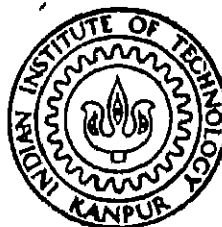


ON WATER TOWERS

By

PRADIP KUMAR KHAN



DEPARTMENT OF CIVIL ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY, KANPUR

OCTOBER, 1988

CE

1988

M

KHA

ON

ON WATER TOWERS

A Thesis Submitted
In Partial Fulfilment of the Requirements
for the Degree of
MASTER OF TECHNOLOGY

By
PRADIP KUMAR KHAN

to the
DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, KANPUR
OCTOBER, 1988

18/10/88

CERTIFICATE

This is to certify that the work towards this thesis entitled 'ON WATER TOWERS', by Pradip Kumar Khan has been carried out under my guidance and it has not been submitted elsewhere for a degree.

October, 1988

A-2 18/10/88

(A. S. R. Sai)
Professor
Civil Engineering Department
Indian Institute of Technology
Kanpur.

ACKNOWLEDGMENT

I am extremely grateful to Dr. A. S. R. Sai for his inspiring guidance, subtle invaluable suggestions, constructive criticism at every stage of my work and being a source of inspiration during the phase of anxiety & uncertainty.

I am grateful to Mr. R. K. Gupta for his adroit suggestions during my work.

I feel bad about leaving the nice company I had here, which made my stay here as a vivacious one. Thanks to Arnab, Chandranath, Debu, Dipankar, E.T., Ginni, Malu, MIM, Mithun, SRC, Suku ...

October, 1988

Pradip Kumar Khan
Pradip Kumar Khan

LIST OF CONTENTS

CHAPTER 1 :	Development of water reservoir system	
1.1	Introduction	1
1.2	Types of reservoir	2
1.3	Cut-and-fill reservoir	3
1.4	Tanks	5
1.4.1	Wooden tanks	5
1.4.2	Steel tanks	7
1.4.3	Masonry tanks	8
1.4.4	Concrete tanks	10
CHAPTER 2 :	Design of water towers	
2.1	Introduction	16
2.2	External appearance	16
2.3	Watertightness	17
2.4	Sketches	18
CHAPTER 3 :	Design of the present water tank	
3.1	List of parts	30
3.2	Data	31
3.3	Section	32
3.4	Idealized section	33
3.5	Force sets for static analysis	34
3.6	Forces for dynamic analysis	36
3.7	Foundation design	37
3.7.1	Design of pile cap below columns	37
3.7.2	Design of pile cap below shaft	44

3.7.3	Design of connecting beam at foundation	46
3.8	Design of columns	47
3.8.1	Column no. 1	47
3.8.2	Column no. 2	50
3.8.3	Column no. 3	53
3.9	Design of shaft	55
3.10	Design of beams	57
3.11	Design of slabs	66
3.12	Design of tank walls	69
3.13	Design of plate at midlanding	71
3.14	Design of stair cases	71
3.15	Revised weight of tank	72
3.16	Estimation of reinforcements	74
CHAPTER 4 : Parametric study of staging of water tank		
4.1	Sketches for idealization	80
4.2	Study cases	81
4.3	Design parameters in columns	82
4.4	Design parameters in bottom circular beam	84
4.5	Design parameters in top circular beam	86
4.6	Design of top roof	88
4.7	Design of bottom dome	89
4.8	Design of top ring beam	90
4.9	Design of bottom ring beam	90
4.10	Design of foundation	91
4.11	Design of columns	91
4.12	Design of tank wall	92
4.13	Calculation	96

4.14	Study of results	98
------	------------------	----

CHAPTER : 5 Optimization of shape of water towers

5.1	Introduction	116
5.2	Previous studies	117
5.3	Formulation of model	119
5.4	Derivation of algorithm	126
5.5	Study of results	128
5.6	Scope of further study	129
6.0	Bibliography	136

LIST OF FIGURES

1. Sketches of water towers	19-27
2. Sections of water towers	28-29
3. Cross section of the water tank	32
4. Idealized section	33
5. Pile-cap below columns	37
6. Section of pile-cap below foundation	40
7. Elevation of pile-cap below columns	40
8. Reactions in piles below columns	43
9. Layout of piles below shaft	45
10. Diagrams showing forces on pile-cap below shaft	46
11. Section of connecting beam at foundation	47
12. Section of shaft	56
13. Section of a typical beam	57
14. Section of liquid retaining slabs	67-68
15. Section of wall for moments	69-70
16. Section of staircase	71
17. Reinforcement detailing for slabs	78
18. Reinforcement detailing for wall	78
19. Tensile forces in bottom circular d.	103-107
20. Axial forces in columns	108-112
21. Percentage of steel in bottom dome	112
22. Variation of tank wall thickness	112
23. Volume of tank wall	114

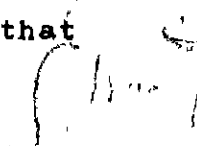
24. Total volume of concrete in tanks	116
25. Section of a typical intake tank	121
26. Sections considered for optimization	122-125

LIST OF TABLES

1. Moment co-efficient for tank wall	69
2. Moments in tank wall	70
3. Moment resistance capacity for slab	70
4. Calculation of minimum thickness of wall at base	93
5. Calculation of minimum thickness of wall at midlevel	94
6. Design of tank wall as tension member	95
7. Volume of tank wall for cases	95
8. List of parameters in shape optimization	126

ABSTRACT

Design of structure is the process by which it is planned so that it will achieve its intended function in safety and at least cost. The advancement of technology has given mankind good structural designs in terms of complicity & sense of beauty, and added a new term 'aesthetics' in engineering design. Water towers are no exception from that. The idea of the present work was conceived when 'LANDSCAPE DESIGNERS ASSOCIATES - N. DELHI' asked for structural design of a flat-bottomed water tower of 2000 Kl capacity supported on eight kinked columns & a central stem at a staging height of 20 m. After designing the structure parametric study of the same for staging have been carried out removing the central stem. The study of water tower design was continued to have a shape having least weight which will be transferred to the staging. A circular intze tank has been chosen for the basic shape and six other cases were considered by changing its various parameters viz., diameters and heights. It gives gave the idea the shape that would be available through the process of optimization.



DEVELOPMENT OF WATER RESOURCES SYSTEM

1.1 INTRODUCTION

In the beginning, people were not concerned with trying to keep water in any particular place. They merely built their various civilization where water was present. Their homes were first trees or caves and later small huts or shelters of varying design. When these were destroyed or damaged by high water, they quickly rebuilt or moved to higher ground. Since homes were simple in any event, people seldom experienced problems with wet or unstable foundations. When contingency occurred, it was easier to move than to solve the problem by ingenuity.

At first settlement were kept close to the water supply, so that their inhabitants could haul water by muscle power - animal or human - utilizing small containers. These served as civilizations first reservoirs. Seepage losses from these small vessels were only a minor problem, as replacement of water was close at hand. As civilization progressed people saw fit to place settlements or locations other than those blessed with water.

The exact time when humanity was first faced with the problem of moving water any appreciable distance is open to some debate. If we believe the writing of PLATO / 21 /, this may well have occurred some 12,500 years ago on Lost Islands of Atlantis. Of course to accept such an account, it is

necessary to believe the existence of such island. At any rate modern historians seemed agreed to one thing : humanity early became concerned with water conveyance and storage because of agricultural needs. That this water was used for drinking to is rather obvious, but it was farming pursuits that started the first primitive undertaking in the field of hydraulics.

1.2 TYPES OF RESERVOIRS

Throughout history, humanity has been faced with the problem of building facilities for storing water and other liquids. As we learned more modern building methods, design changes and other alterations were made as new materials became available. Although initial construction efforts were primitive by today's standard many structures built by ancients are still standing, and it is interesting to note that three out of four basic designs used today for water storage were developed thousands of years ago. Of the type listed below, only steel tank is a relatively modern development. The introduction of reinforcement steel into concrete, however did not change the basic concept of concrete tank : (in) merely improved the design.

Despite many variations, only two general methods are used in designing large water holding facilities. As the following tabulation shows, there are four variation of tanks, and the last of these may further be classified in three ways :

1. Cut-and-fill reservoirs

2. Tanks

- A. Wooden

- B. Steel

- C. Masonry

- D. Concrete

- i) Cast

- ii) Prestressed

- iii) Wire wrapped.

1.3 CUT AND FILL RESERVOIRS

In this construction method, a depression with slopping side is created by digging and removing the earth with appropriate equipment. The earth so removed is utilized to construct fill or embankment around the top of the excavation to increase the effective height of the water and to reduce excavation costs. Part of the water is held below the original ground level (in cut) and part above this level (in fill). The slopes and the bottom of the excavation are then lined with appropriate material to prevent loss of water by seepage and also to protect the earthen component of the facility. Even when excavated material is not used in the actual construction, the design is still referred to as cut-and-fill reservoir.

The most important advantage of cut-and-fill reservoir is its basic low cost. This is accompanied by a low profile,

an important consideration when the reservoir is to be constructed in a newly developing area when accent is on beauty. The structural strength of the facility comes from the compacted earth itself, which can be formed into place more economically than steel, concrete, or other materials of construction. No other type of reservoir can be built as economically as this one. } P₀

Unfortunately, this seemingly ideal water holder is not without its disadvantages. Although the earth is really holding the reservoir together, it must do so without fail. The technical aspects associated with this requirements are not assembled in any one place for easy access by the engineer. An engineer who has never built a cut-and-fill reservoir has considerable homework to do. Several disciplines are involved and the engineer must become familiar with all of them. In contrast, steel and concrete tanks are predesigned by various marketing organizations, and the engineer using the type of storage facility need only ^{to} copy this standard designs. ?

Moreover, unlike the steel, wooden or concrete tank, the cut-and-fill reservoir cannot hold water in vertical configuration. It must utilize the side supports that has slopes. A protective fence should surround the reservoir. Its specification are generally dictated by various agencies responsible for safety features of open storage facilities.

If the reservoir is holding potable water, either raw

or finished, additional safety features are required. Current practice is to build a fence on the outer embankment, so that the bottom of the fence is at least 8 ft. below the berm's top elevation. The fence is usually topped with various arrangements of barbed wire to further discourage entry.

1.4 TANKS

1.4.1 WOODEN TANKS

Wood is a very a popular material for use in water tank construction in early 1900s, and it continued well unto 20th century. At that time most of the water tanks used for railway water service were constructed of wood, and their use in this area did much to spur the growth of the wooden tank industry (21). At present the rate of building new wooden tanks diminished considerably because of economic and technological disadvantages of the type of holding systems. Wood construction does offer some advantages for small tanks but the decreasing demand for such tanks has likewise been a factor in the decline of wooden tank business.

Seventy years ago, wood offered several advantages in tank construction. At that time, concrete and steel were not very common, there was a decided lack of technology in connection with their efficient use. In contrast wood wood working was a long-practiced art, and the joining and fitting of the staves were done with great skill. The staves were produced in the factory and properly coded and numbered

for easy assembly in the field / 21 /.

Aside from a change in tensioning hoops from flat bands to round rods, little change in construction procedure has been made in the years. The staves varied in thickness unto about 3 inch., depending on the size of the tank. A groove was cut at bottom of the staves for receiving the edges of the planks that made up the bottom of the tank.

Hoop tension was achieved by tightening nuts on specially designed lugs. Tensioning could be increased at any time, merely by additional turning of the tightening nuts. Many early failures resulted when the originally designed flat tensioning flats were used. Corrosion of the hoops next to the tank went unnoticed until tank literally fell apart. A change to rods with circular cross section did much to eliminate this problem.

On the credit side of the ledger, wood is free from the rust problem that plagued steel construction at the time, and wooden tank has certain inherent protection against frost and cold weather, which cause problem to the concrete tank. When the fibers of the wood in the tank is thoroughly saturated decay is practically impossible. Also if the wood is perfectly dry, there is little chance for decay. It is the in-between condition that cause the wood to deteriorate. Even then the wooden tank has a respectable life span, especially if the better aging timbers are selected.

1.4.2 STEEL TANKS

Although historians disagree as to the exact time when iron was first smelted the consensus of opinion places at the event at about 1500 B.C. Despite this early beginning, things went slowly, as the technology of steel making proved to be very complex. In addition the massive equipment and enormous power is required to roll steel into plates were slow of attainment.

In short, the jump from iron to steel did not happen overnight. Again history is somewhat vague as to when this took place, and its occurrence is further being crowded by the complexity of the change. High-quality steel was produced in the ancient past, and in 384 B.C. Arristotle described its manufacture. The steel was made in India and the metal shipped to Damascus where it tempered and hammered into sword of "Damascus Steel" / 21 /. For steel to be useful in the fabrication of all-steel water tanks, it must be of plate form. This requires that the steel ingots to be rolled into plane sheets. Although evidence shows that hot rolling of steel took place in the late 16th century, the process was crude and yielded steel plate of very poor dimensional tolerances.

As the steel industry learned how to roll plates into thin sections, it began to look into potential business in the waterworks field. Quick inroads was made into this area because of apparent advantage of steel-type construction

over concrete and wood. No spalling, no cracking, and no problems at wall-floor junctions were of the selling points. The industry also boasted that the standard designs could be furnished to the engineer and this did an effect in the speeding up of the specification process. On first cost basis, steel tank seemed to offer decided advantages over its concrete competitor, particularly in the smaller sizes.

It soon became apparent, however, that the steel tank did not provide the low-cost carefree facility predicted by the seller and hoped for by the owner. Corrosion, then as today, was a very serious problem. It strikes on four major fronts: (1) where the steel is in contact with the water, (2) where the steel is in contact with the moist air above the waterline, (3) in the foundation areas where the steel touches the soil and, (4) in areas of contact of dissimilar metals.

1.3.3 MASONRY TANKS

The masonry tank for water storage was a popular form of construction at one time. In fact, the further back we go in history, the more popular it was / 21 /.

To compensate for their structural weaknesses, very early tanks built on the principle that any destructive force could be overcome if enough mass was designed into the wall. Such was sound engineering logic in ancient times when rulers had an abundant supply of low-cost slave labor. It is

true that the slaves had to be housed and fed, but these costs were kept to an absolute minimum. Such a system for funding was hard to beat. It eliminated the need for cost control and value engineering since no one could improve on the cost of slave labor.

From seepage control standpoint, too, masonry construction represented sound engineering. The building blocks were large and dense, and the resulting mortar path between them was a long one. Hence initial seepage was relatively low. Later, or sometimes immediately after construction, bitumen-type linings were applied to the inside of the tank. Actually, they served as primarily construction joint seals, rather than linings because of the impervious nature of the rocks used.

Today high labor costs and modern design has eliminated masonry tank construction except in very special cases. Masonry is difficult to reinforce, yet because of its poor structural stability such reinforcement is generally required in the construction practice. Early tanks of the present era were often built of rough stones, with even rougher texture mortar joints. This surface condition translates into an expensive cost of preparation of the wall to receive any future lining system. For this reason it is not economically feasible to line small masonry tank. Tanks that leak are usually abandoned or operated at reduced water levels.

In its limited applications today, masonry construction for water holding facilities generally makes use of cement or cinder blocks. This approach eliminated formwork and claimed to be a less expensive method of construction. Walls of limited height and are often extensions of existing facilities for the purpose of increasing their capacity. Some new facilities have been built using block construction to increase the holding capacity of cut-and-fill reservoir designs. In both cases reinforcement is used to attain necessary structural stability.

The block design mentioned above requires a lining of some type to keep seepage loss within respectable limits, since the blocks are quite porous. During construction extra care must be used to ensure that the mortar joints are given a smooth finish, flush with the inside face of the blocks. This is particularly important if thin membrane system are to be utilized as the lining.

With the advent of steel and modern concrete tanks, The masonry design rapidly declined in favor. Their main claim to fame is their early origin. They also played an important role in transitional development patterns of the concrete tank.

1.3.4 CONCRETE TANKS

The concrete tank as we know it today was developed through a process of evolution. At one point in the process,

the development was actually reversed itself at the direction of the cut-and-fill reservoir. Early tanks made limited use of cement. The cement was made up into a mortar and used between stones to construct vertical tank wall. Thus the reservoir was built-up type of structure. As a finishing operation a thin plaster mix was sometimes troweled onto the resulting surface to give added watertightness and a more pleasing appearance. This type of construction did the intended job when seepage losses were tolerated as a necessary evil and when labor costs were low. Walls were very thick by today's standards, often exceeding 12 inch.; 2 ft. wall sections were not at all uncommon.

Early technology centered around the vertical wall structure that is referred to as tank. This is not surprising , since the above-grade tanks are cousins to cisterns and wells. The building of later structure is an ancient art, so the adoption of built up masonry construction to above-grade tanks was a logical step. It was also a step that could be taken piecemeal, in that portion of the tank could be set into the ground and the remainder extended into the air. As experience was gained, higher tanks could be constructed, thus decreasing unit storage costs.

The development of portland cement was closely associated with waterworks facilities. But the use of portland cement in waterworks facilities had a very shaky

beginning, because the original block fell into pieces when immersed in water. Troubles were due to improper mix and curing conditions, lack of proper reinforcement steel, design inadequacies, and a generally poor knowledge how concrete and its reinforcement react to various environments. With the advent of more effective communications and the development of trade associations many problems have been solved. It is still necessary, though, to conform to good design practice coupled with high-quality construction standards and to evaluate carefully all factors that influence the final overall design.

Before reinforcing steel was available or its use was well understood, the designer, if he or she went into air with the tank, could logically do so only by employing masonry tactics. To be on the safe side, original designs included heavy wall sections. This feature was copied later by most designers, as innovation was not a popular trend. Designers and builders stuck to things that had worked in the past.

The designers had the idea of placing retaining wall on the sloped earth, allowing the later to support the tank walls. In this approach the tank is no longer a tank, but becomes a reservoir. The "walls" were still brick and mortar, rock and mortar, or, later, unreinforced poured concrete. Combinations were also used, particularly ones utilizing a concrete overlay on top brick or rock

construction. Exactly when this construction principles were first utilized in modern times is unknown, but many facilities of this type were built in the years before 1920.

The concrete evaluation process, which seemed to culminate in the cut-and-fill reservoir, did not stop there. Instead it reversed itself and went back to the vertical direction whence it came. This happened for a number of reasons. Excavation methods were crude and costly. Trucks did not exist, nor did heavy earthmoving equipment. Also, though a less of a problem then than now, the cut-and-fill reservoir required considerably more ground space than did the tank. For a while, however, the later argument was counterbalanced by the fact that the building materials, construction techniques of the day, and psychological factors somewhat limited the height of the tank construction.

During this period of evolution, formed and poured concrete tanks made their appearance. Again walls were very thick. Even though reinforcements were introduced, it was crude in comparison to that used today. It went to both extremes, being either thin wire mesh or very heavy, twisted cross sections. The concrete quality varied greatly. Some of this structures are still in excellent condition, the concrete being hard and dense, whereas in others the concrete is of very poor quality, appeared to have suffered because various combinations of poor mixing, low cement

content, or chemical degradation processes. Cracks in the structures were common, this being a particularly serious difficulty with respect to cold pour joints. Builders were not familiar with the problem, and cement mixing which was done by hand, could not keep up with efficient pouring schedules.

Tanks were often set into the ground, or at least a major portion of their depth was in the ground. For this reason seepage went unnoticed or it was just that no one bothered with remedial measures. On some old tanks, though, evidence of crack repairs using concrete and asphaltic materials is visible.

There was a difference of opinion with respect to overall design of early concrete structures. One school of thought decreed that the structures should be built with vertical walls, whereas another believed that the sloping walls were better. The later group argued strongly. Formwork was eliminated, they said, and reinforcement was not needed. Although these things represented only a small money savings by today's standards, they still accounted for appreciable percentage of the total job cost. An equally enticing advantage of sloped wall construction was its apparent simplicity, both for the designer and the constructor. Of course, they were soon to find that these tanks, more designated as cut-and-fill reservoirs, present a batch of problems of their own. These center around the subject that how to keep the water inside the structure from leaking out

of it. Builders of vertical-wall tanks were well aware of leakage problems in their creations, and they seemed to find some comfort in the fact that they were not alone in this respect.

To cut the cost and improve the quality of concrete tanks, reinforcement steels were added. This reduced the concrete mass by absorbing and restricting tension loads. During this period, construction of concrete tanks evolved in two directions : (1) the reinforced tanks, and (2) the prestressed tanks.

DESIGN OF WATER TOWERS

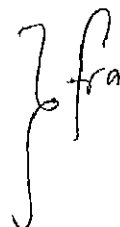
2.1 INTRODUCTION

Water towers are usually constructed of in situ reinforced or prestressed concrete and generally consist of an elevated tank supported on columns or on a massive central stem or combination of both. The older, plain, square or rectangular concrete tanks on four columns have in recent years given place to very elegant and attractive structures. In some instances, particularly on the Continent and overseas, the tank is cast at ground level and then jacked up the central stem.

2.2 EXTERNAL APPEARANCES

Water towers are normally conspicuous structures as they are located on relatively high ground and in addition they are elevated.

The engineer concerned should cover the aesthetics of design of structures and some people will no doubt consider that this is the province of the architect rather than the engineer. It is suggested that good engineering should include a design which is aesthetically satisfying. The great engineers of the past have always achieved this. One of the many advantages of concrete as a construction material is the variety of external finishes which can be obtained.



When considered the type of finish, the problems of weather staining must be given close attention and careful detailing is needed for an otherwise pleasing structure is not to be spoilt by unsightly streaks and patches. There has been a move away from smooth, plain finishes to a rougher type of surface, such as a broad-marked, exposed aggregate and sculptured. When considering the type of finish required, it is important to remember that the mix design chosen must be suitable for watertightness as well external appearance.

2.3 WATERTIGHTNESS IN TANKS

Nothing spoils the appearance of an otherwise attractive water tower more than seepage of water through cracks, joints, and defective concrete. It is not simple matter to design and construct a large water tank at a height of 20-40m. above the ground, so that it is literally 'bottle' tight. A lower standard of watertightness can cause disappointment from an appearance point view.

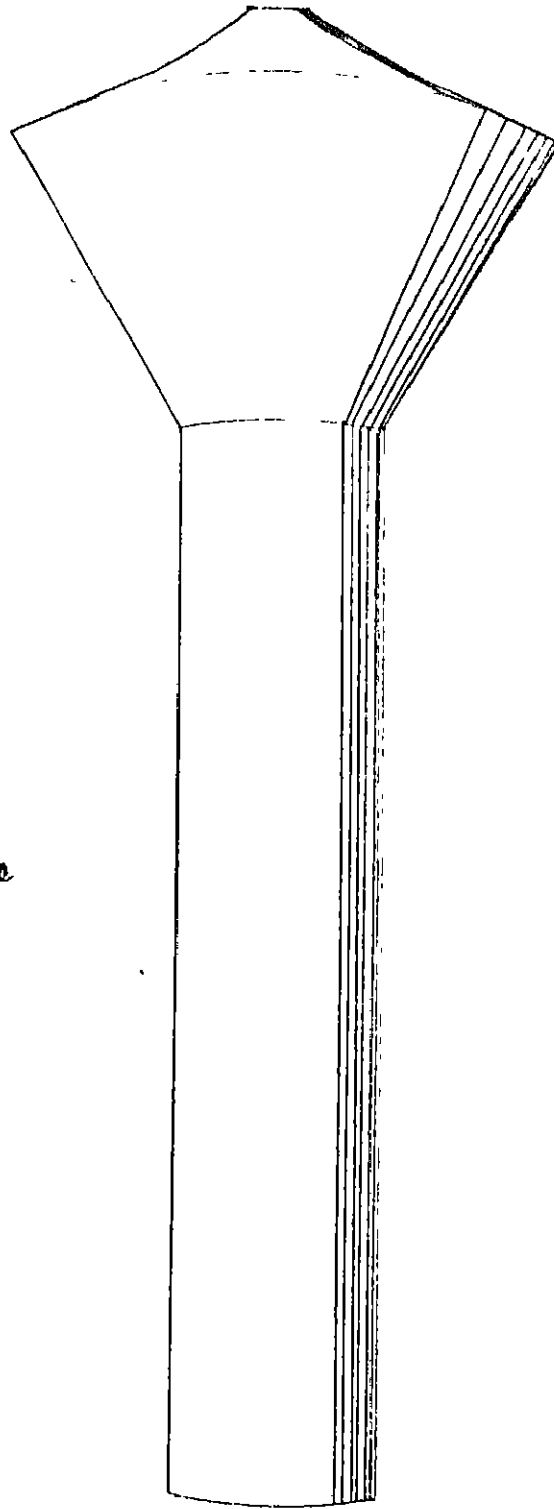
? It is impossible to construct a structure of any size without joints and in a water tower these will be visible. Therefore they should be featured and form part of the external design, and must be completely watertight. With in situ reinforced concrete water lining should be given careful consideration.

The lining can be either performed material, such as PVC or polyisobutylene (PIB), or in situ coating based on polyurethane and epoxide rasins / 1 /. For towers containing potable water, the lining must be non-toxic and non-taining, and must not support the growth of bacteria, fungi and algae.

2.4 SKETCHES

A few sketches of water tanks are here shown here which are being used in practice / 1,2,3,6,17 /. At first full sketches are drawn & then sectional views has been shown here.

fram



should have
specific ~~name~~ name @/title

Fig 1a. - Sketch of water tower

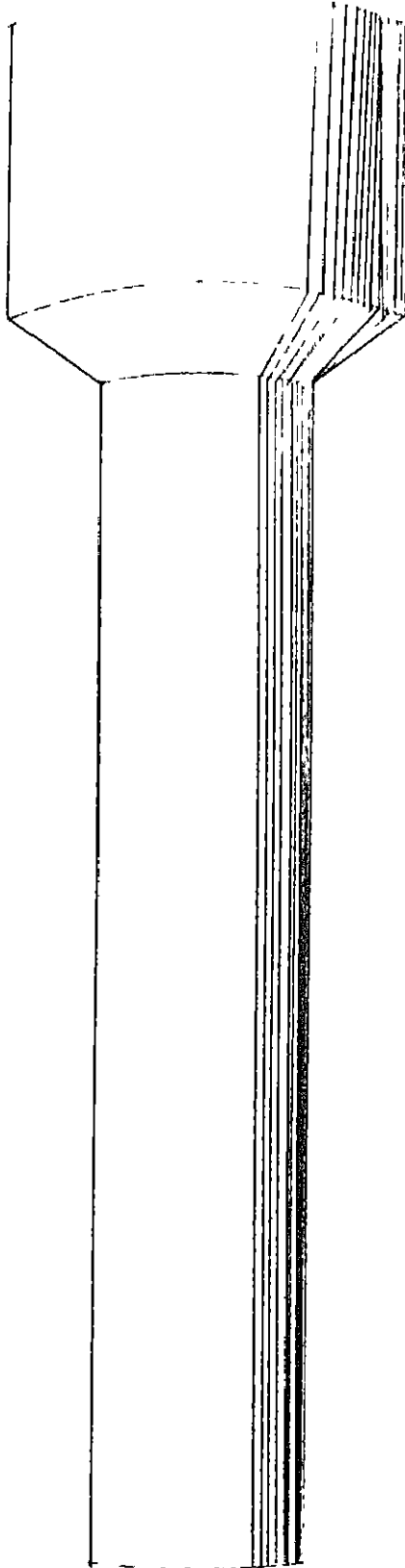


Fig. 1b. - SKETCH

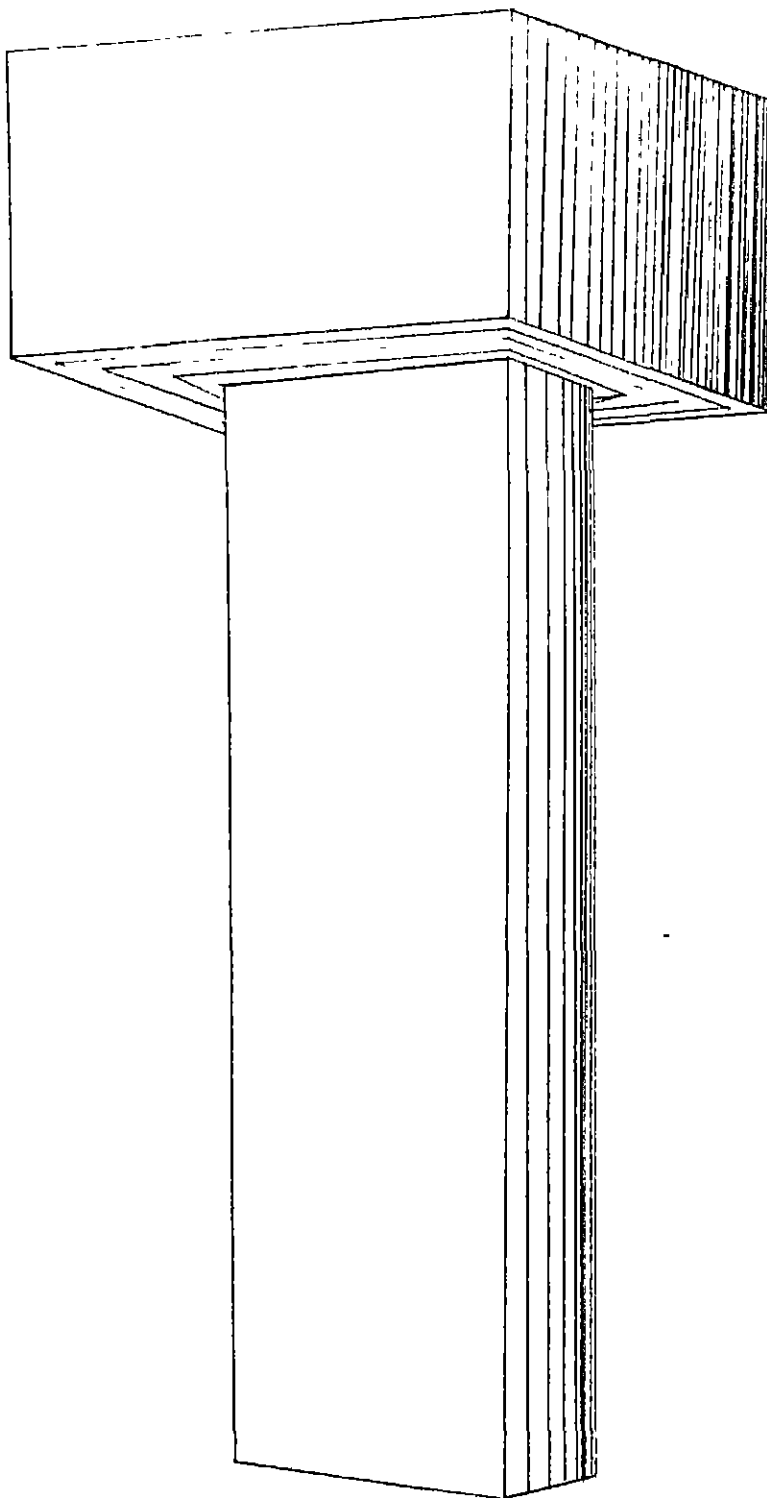


Fig-1C - SKETCH

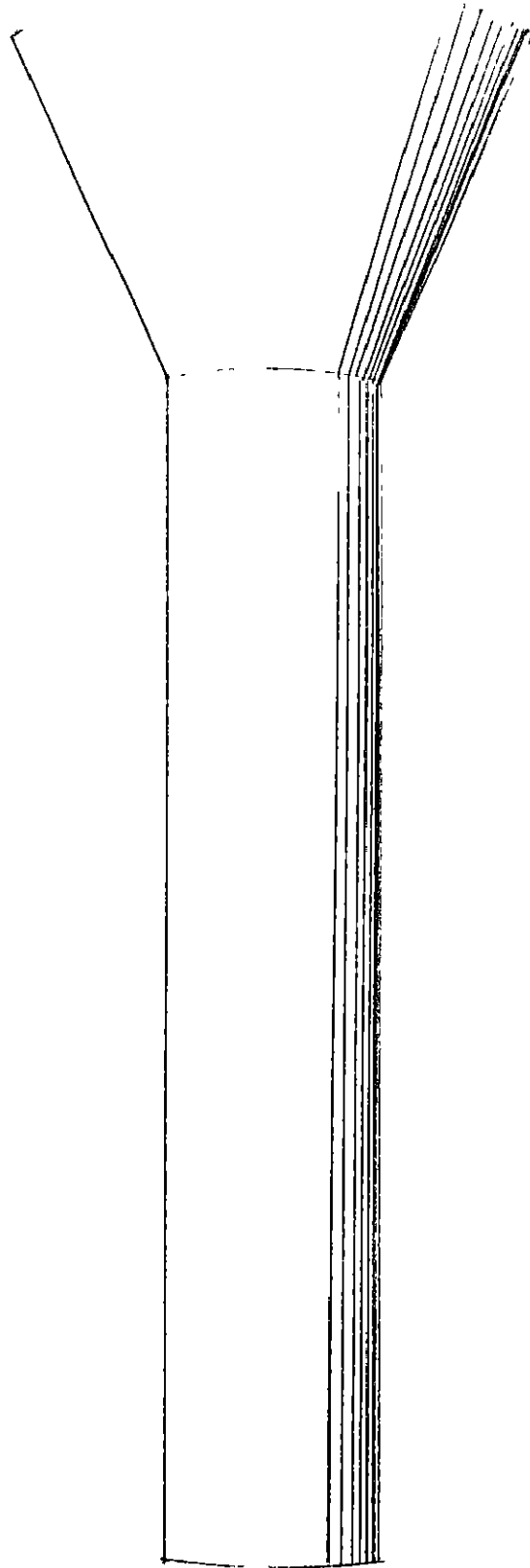


Fig.-1D- SKETCH

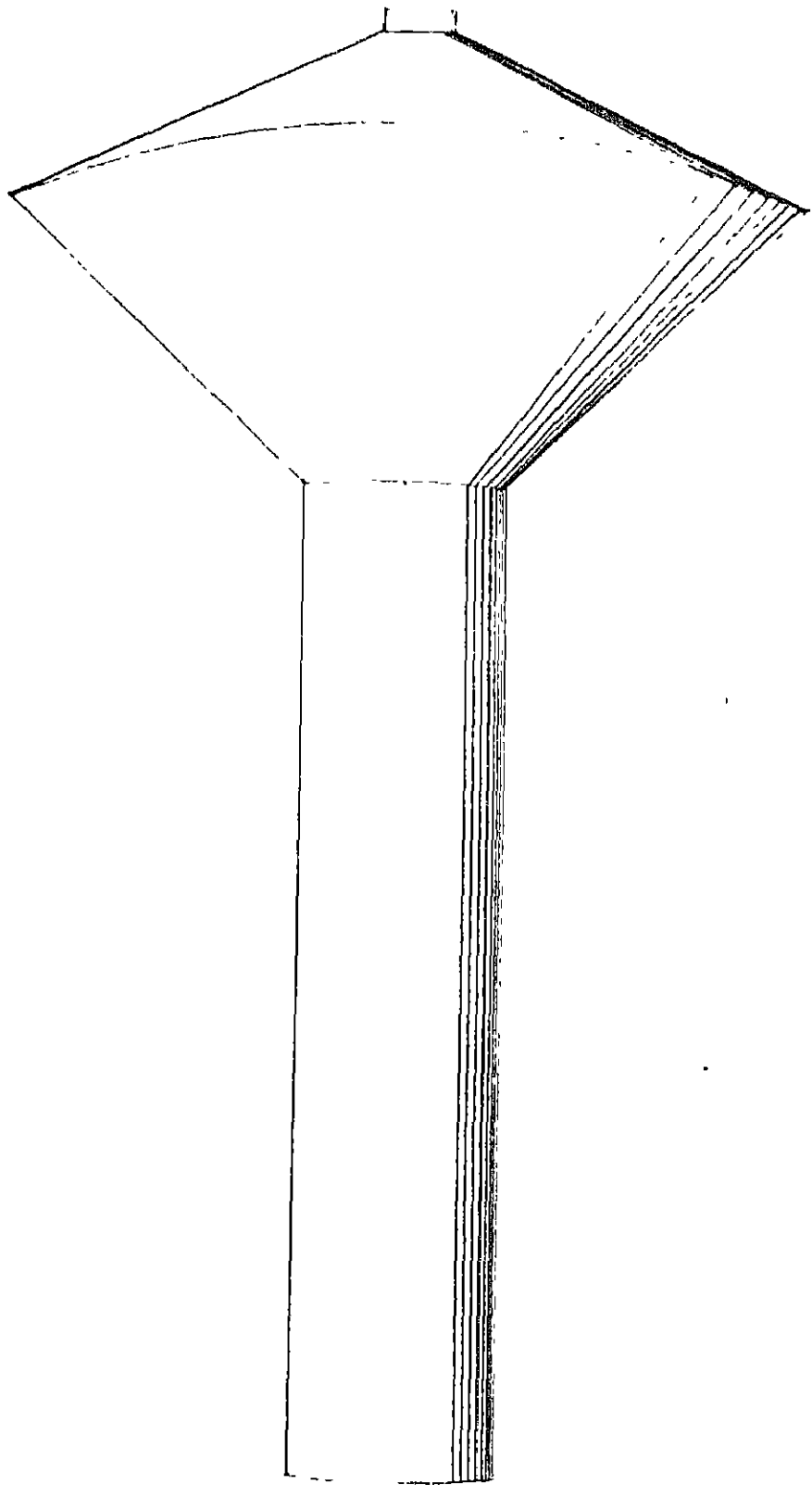


Fig - 1E - sketch

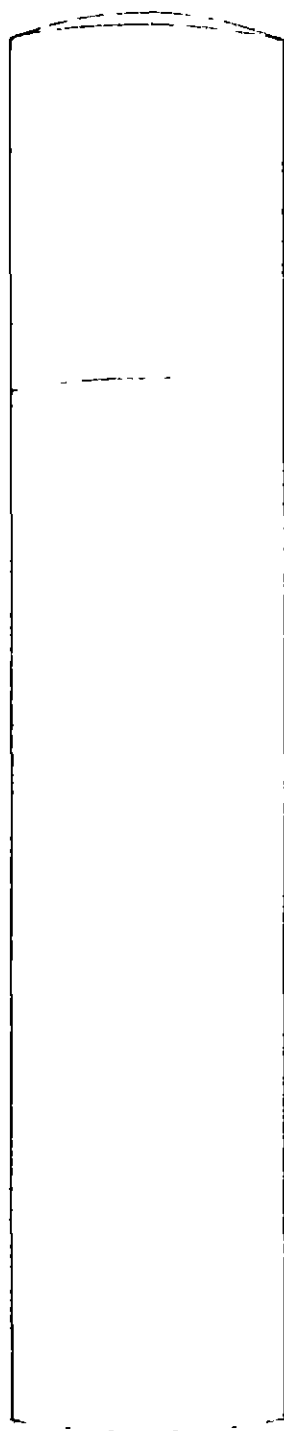


Fig- 1F - SKETCH

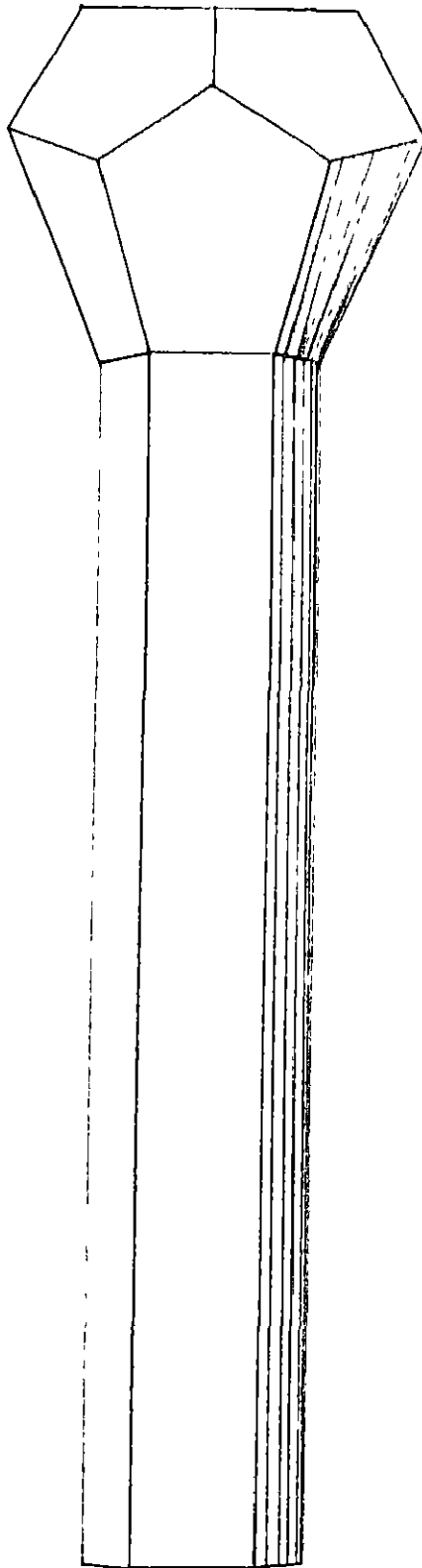


Fig-16 - SKETCH

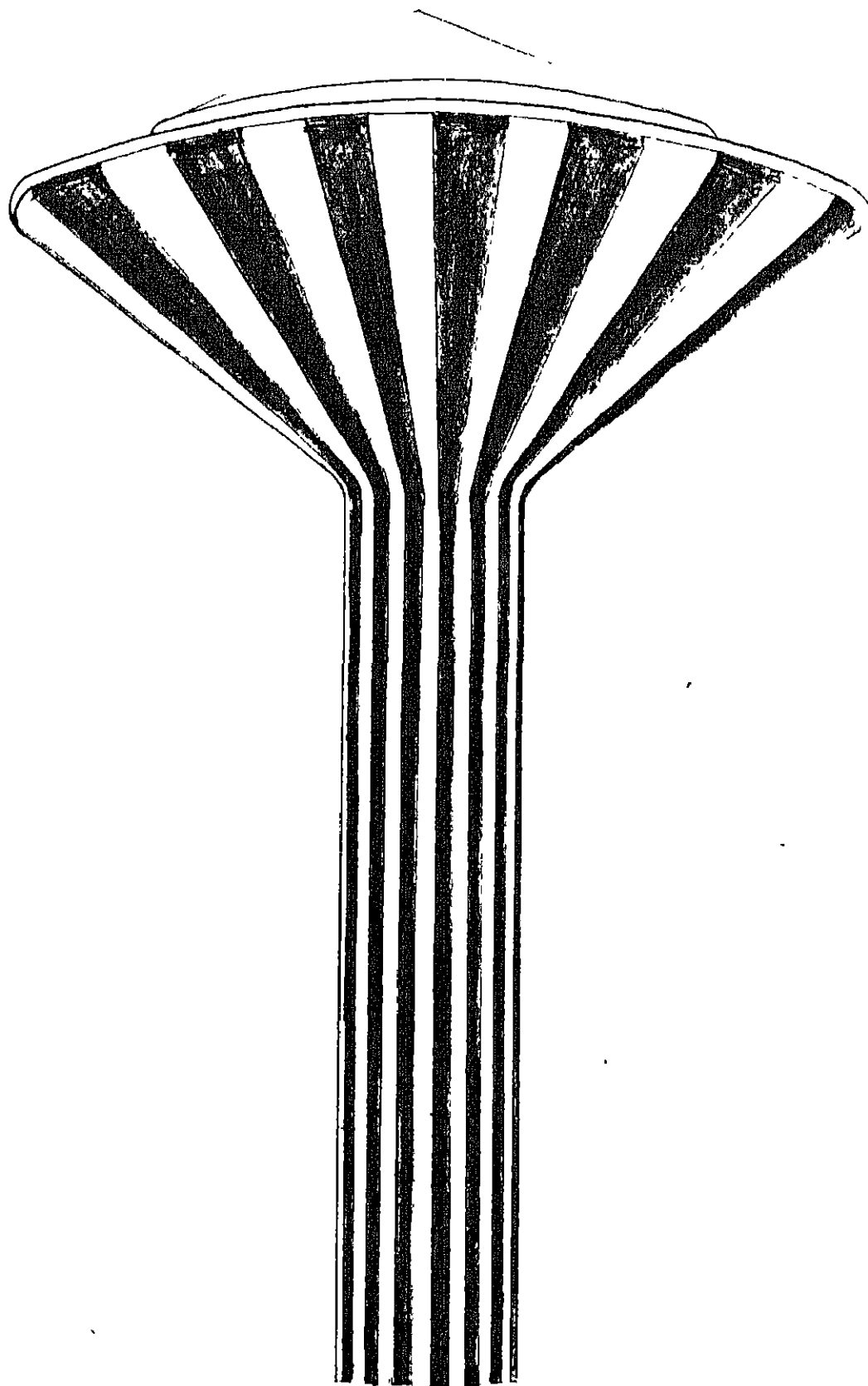


FIG. 1 H - SKETCH

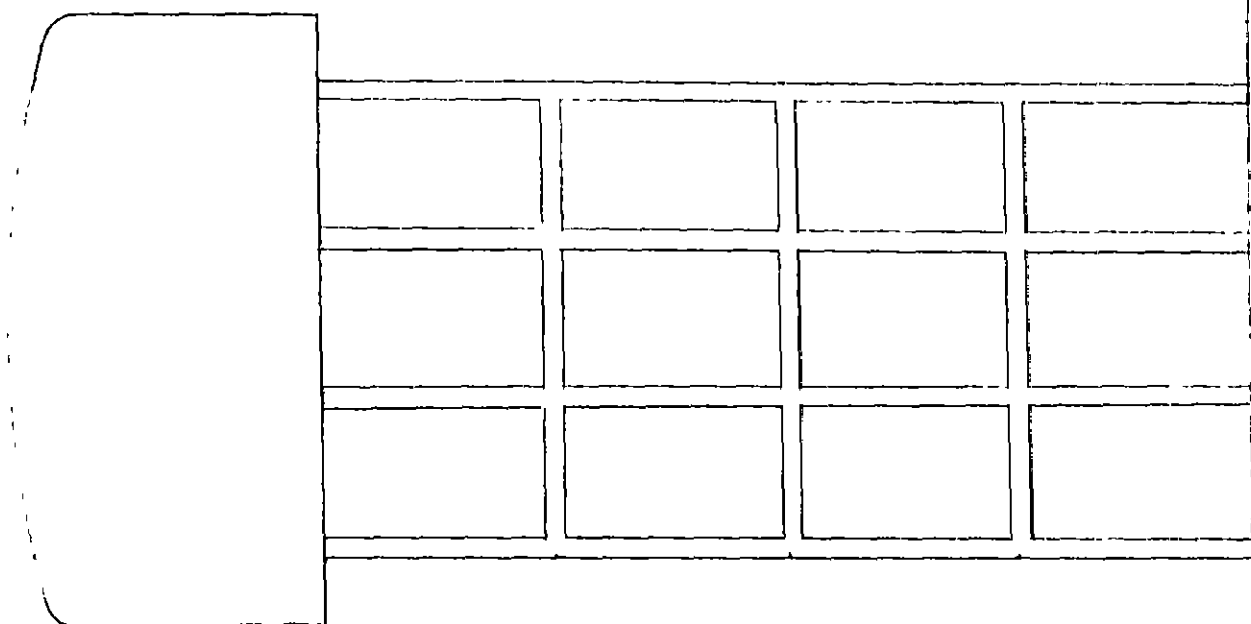


Fig. IJ - SKETCH

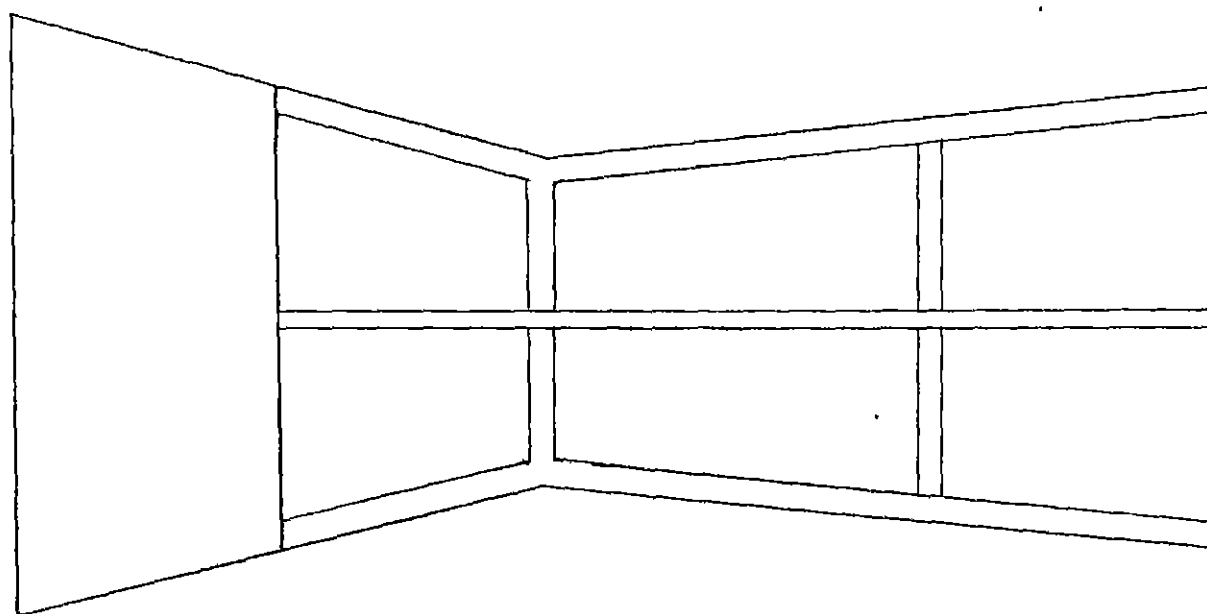
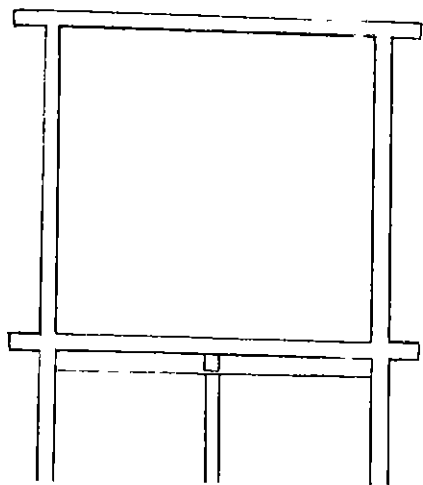
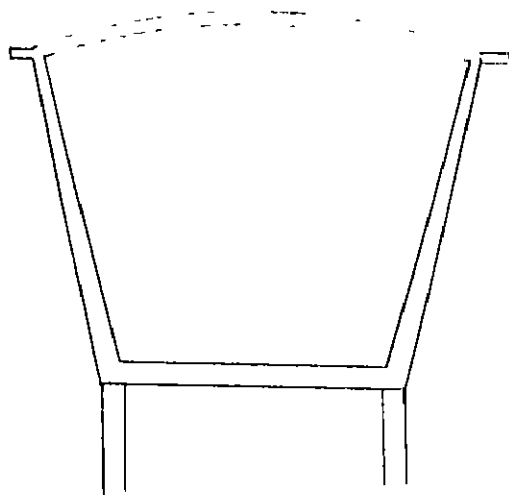


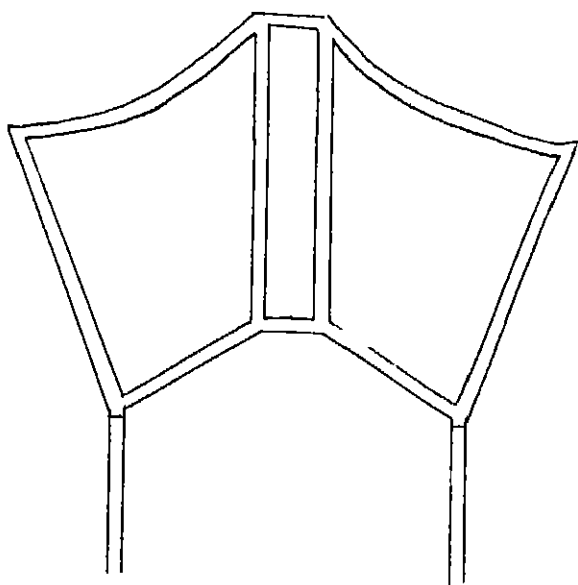
Fig. II - SKETCH



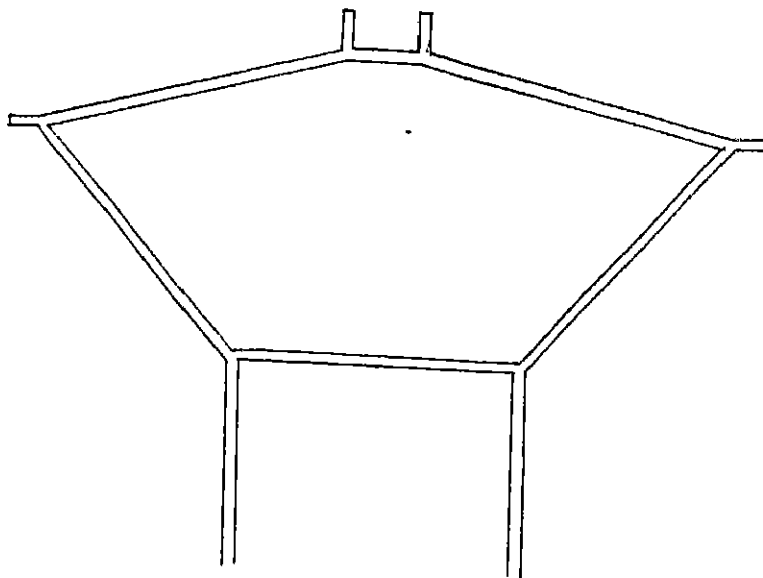
(A)



(B)

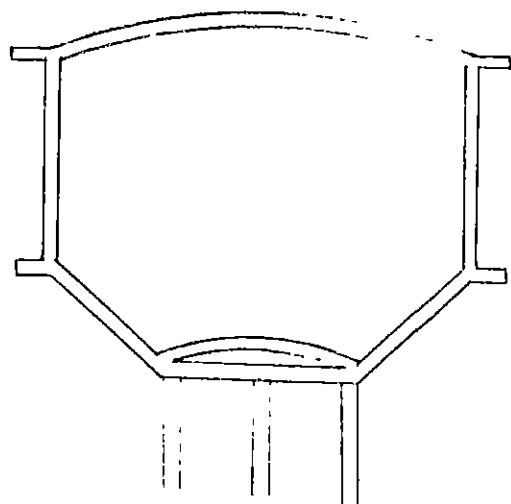


(C)

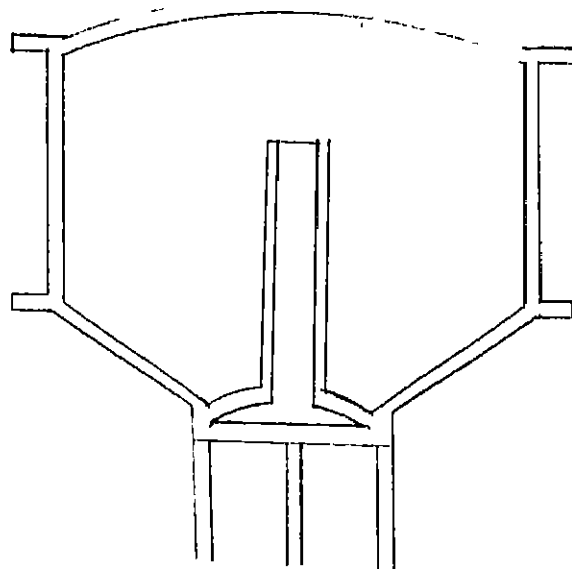


(D)

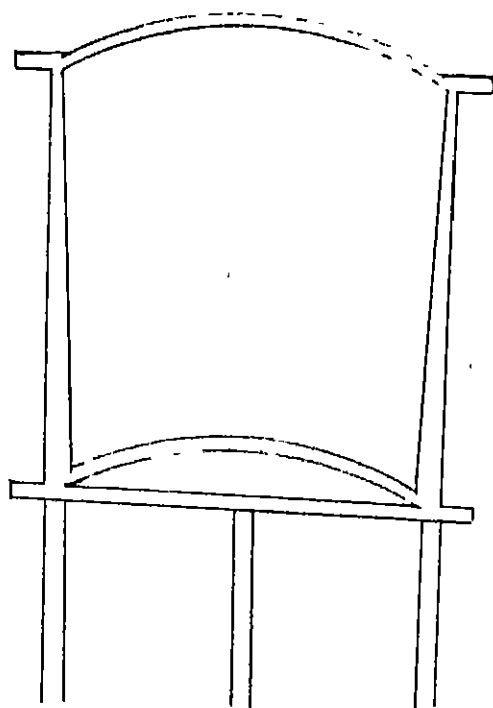
Fig 2. sections of watertower



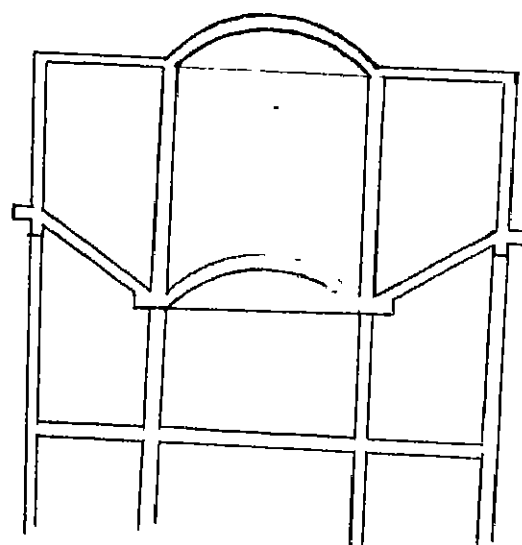
(E)



(F)



(G)



(H)

Fig 2- section of watertowers

3.1 Design of Water Tank ::LIST ::

3.1.1 SUBSTRUCTURE

- A) Below Columns:F1
- B) Below Shaft:F3
- C) Connecting beam between column & shaft foundation:F2

3.1.2 SUPERSTRUCTURE

- A) Columns
 - 1) Level 1:CL1
 - 11) Level 2:CL2
 - 111) Level 3:CL3
- B) Shaft:SH
- C) Tank wall:TWL
- D) Slabs
 - 1) Top part:SL1
 - 11) Bottom part:SL2
- E) Beams
 - 1) Top slab
 - a) Radial beams:RB1
 - b) Circuler outermost beams:CB1
 - c) Circuler intermediate - 1 beams:CB2
 - d) Circuler intermediate - 2 beams:CB3
 - e) Circuler innermost beams:CB4
 - 11) Bottom slab
 - a) Radial beams:RB2
 - b) Circuler outermost beams:CB5
 - c) Circuler intermediate beams:CB6
 - d) Circuler innermost beams:CB7
- F) Plate at midlanding:PLT
- G) Staircase:STC
- H) Dome at top:DME

*(Printed) Slab
the ref. w. Fig. etc.
to make out
the otherwise, hand*

3.2 DESIGN OF WATER TANK

DATA :

Capacity	=	2000 KL
Free board	=	0.30 m.
Staging height	=	20.0 m.
Shaft radius (outer)	=	4.00 m.
Shaft thickness	=	125 mm.

Inner diameter of water tank = 24.0 m.

Depth of water in tank = $2000 \times 4 / \pi \times (24^2 - 4^2)$
 = 4.547 m. \approx 4.6 m.

Total height of container = 4.90 m.

Let depth of slab = 0.3 m

So self weight = 7.5 KN/m

Weight of water = 46.0 KN/m

Total weight = 53.5 KN/m

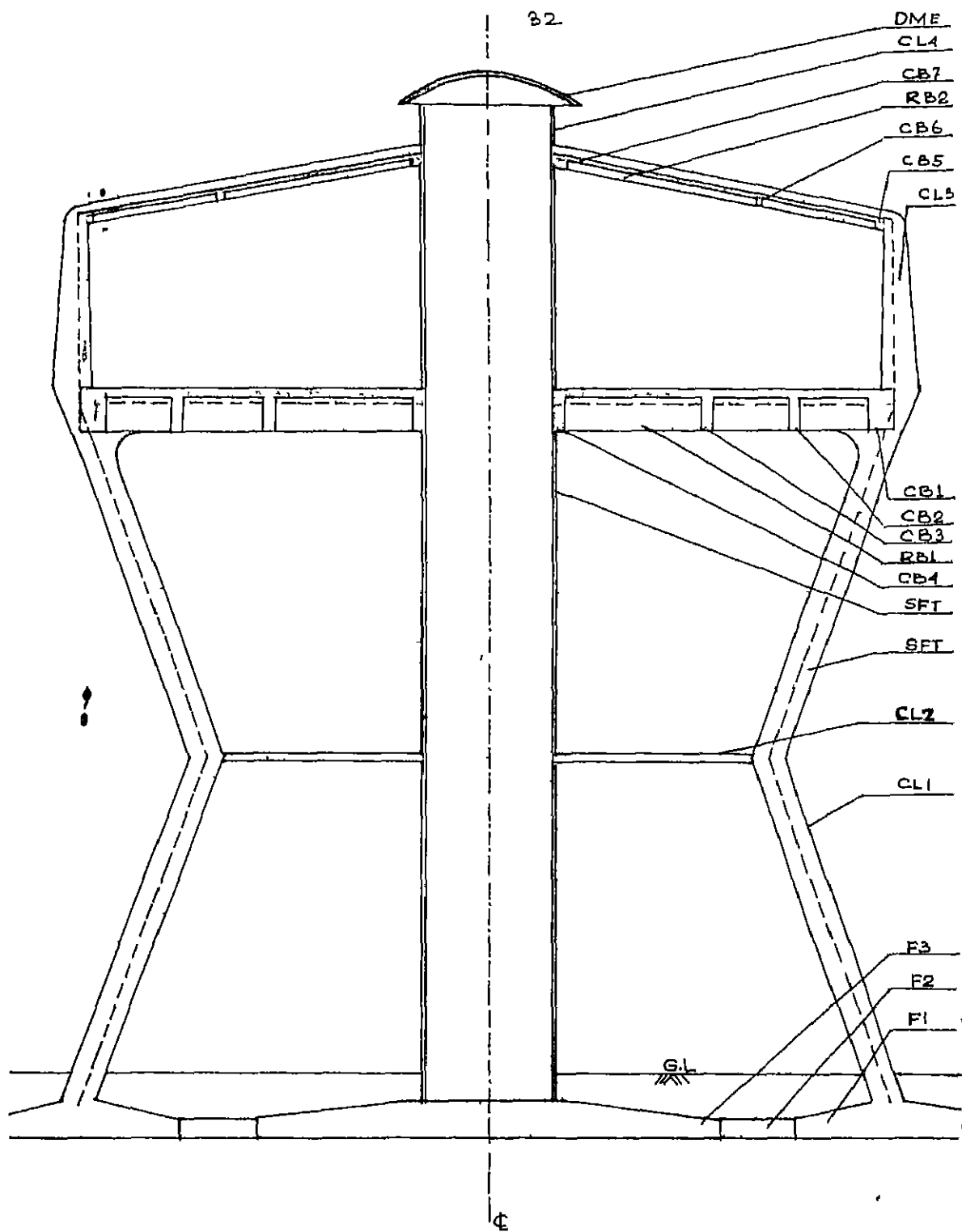


Fig -3 CROSS SECTION OF THE TANK

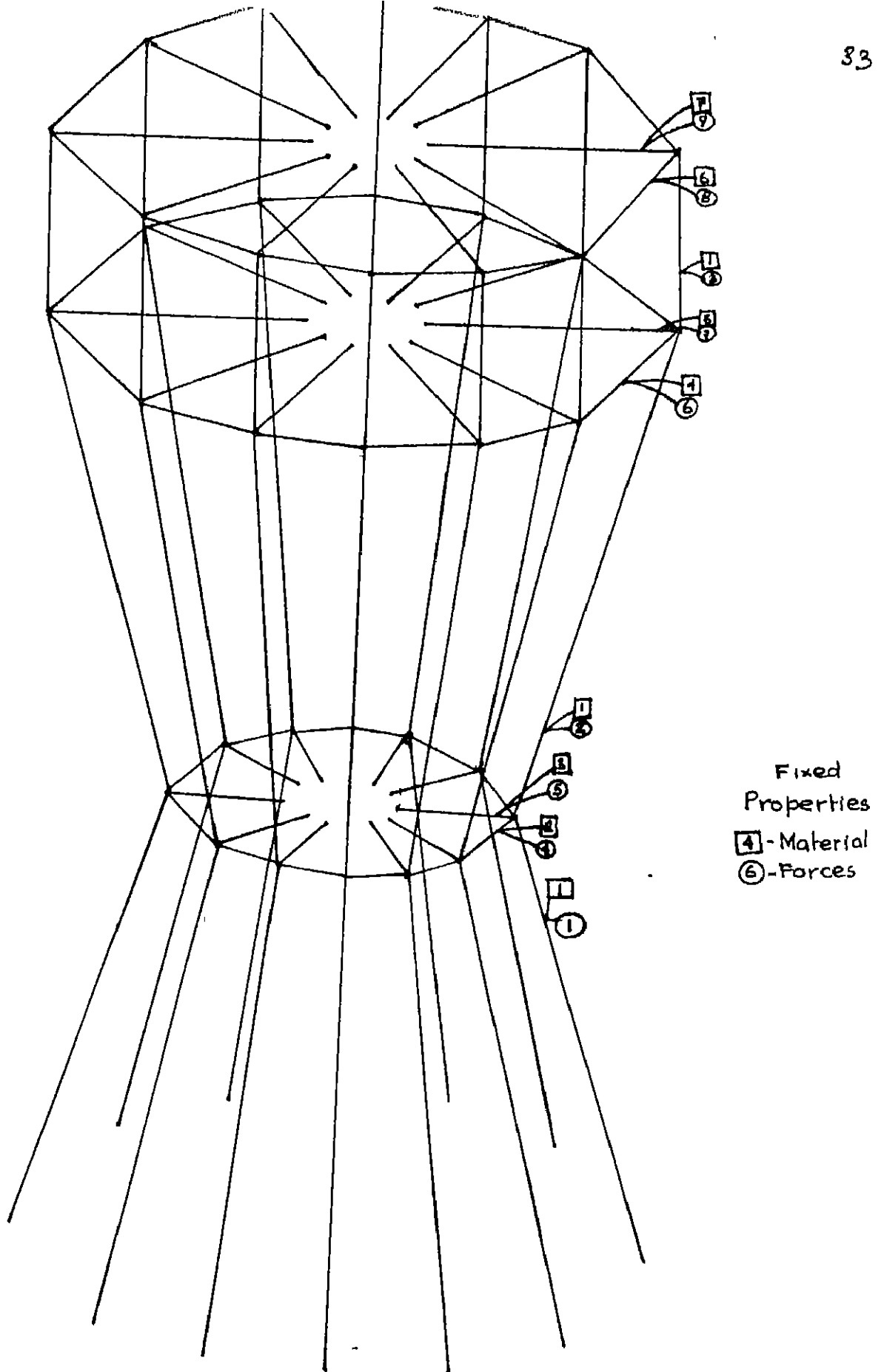


Fig-4 IDEALIZED SECTION OF TANK FOR ANALYSIS

3.5 STATIC ANALYSIS :: Fixed end force sets ::

No. : 1

$$\begin{aligned}\theta &= \tan^{-1}(10.0/3.5) = 70.710^\circ, \quad \omega = 1*0.5*25 = 12.5 \text{ KN/m} \\ \omega_d &= \omega/\cos\theta = 37.84 \text{ KN/m}, \quad l = 3.675 \text{ m} \\ M &= \omega_d l^2/12 = 42.586 \text{ KNm}, \quad \omega_s = 22.969 \text{ KN} \\ \omega_a &= \omega \tan\theta = 35.71 \text{ KN}, \quad \omega_4 = \omega_a l/2 = 65.625 \text{ KN}\end{aligned}$$

No. : 2

$$\begin{aligned}\theta &= \tan^{-1}(10.8/3.78) = 70.71^\circ, \quad \omega = 1*0.5*25 = 12.5 \text{ KN/m} \\ \omega_d &= \omega/\cos\theta = 37.84 \text{ KN/m}, \quad l = 3.78 \text{ m} \\ M &= \omega_d l^2/12 = 45.054 \text{ KNm}, \quad \omega_s = 23.625 \text{ KN} \\ \omega_a &= \omega \tan\theta = 35.71 \text{ KN}, \quad \omega_4 = \omega_a l/2 = 67.857 \text{ KN}\end{aligned}$$

No. : 3

$$\begin{aligned}l &= 5.65 \text{ m}, \quad a = 0.50 \text{ m}, \quad b = 0.55 \text{ m}, \quad s = 4.60 \text{ m} \\ \omega &= \text{Triangular water pressure of height } 4.60 \text{ m.} \\ M_i &= 441.557 \text{ KNm}, \quad M_j = 285.344 \text{ KNm} \\ R_i &= 425.513 \text{ KN}, \quad R_j = 239.227 \text{ KN}\end{aligned}$$

No. : 4

$$\begin{aligned}l &= 4.451 \text{ m}, \quad \omega = 0.6*0.4*25 = 6.0 \text{ KN/m} \\ M_i &= -M_j = 9.904 \text{ KNm}, \quad R_i = -R_j = 13.352 \text{ KN}\end{aligned}$$

No. : 5

$$\begin{aligned}l &= 6.5 \text{ m}, \quad \omega = 0.6*0.5*25 = 7.5 \text{ KN/m} \\ M_i &= -M_j = 30.14 \text{ KNm}, \quad R_i = -R_j = 24.375 \text{ KN}\end{aligned}$$

No. : 6

$$\begin{aligned}
 l &= 6.178 \text{ m}, \omega_{\text{self}} = 32.5 \text{ KN/m}, \omega_{\text{wall}} = 18.375 \text{ KN/m} \\
 \omega_{\text{slab}} &= 9.0 \text{ KN/m}, \omega_{\text{water}} = 46.0 \text{ KN/m (Trapezoidal)} \\
 M_i &= -M_j = 344.853 \text{ KNm}, R_i = -R_j = 309.59 \text{ KN}
 \end{aligned}$$

No. : 7

$$\begin{aligned}
 l &= 10.0 \text{ m}, \omega_{\text{self}} = 12.81 \text{ KN/m (u.d.l.)} \\
 \omega_{\text{water+slab}} &= 336.141 \text{ KN/m \& } 56.015 \text{ KN/m (at node i\&j)} \\
 P &= 45.32 \text{ KN}, P = 30.30 \text{ KN (point load)} \\
 M_i &= 2070.3383 \text{ KNm}, M_j = 1570.6216 \text{ KNm} \\
 R_i &= 1325.3583 \text{ KN}, R_j = 839.1617 \text{ KN}
 \end{aligned}$$

No. : 8

$$\begin{aligned}
 l &= 6.178 \text{ m}, \omega_{\text{self+wall}} = 24.625 \text{ KN/m} \\
 M_i &= -M_j = 78.323 \text{ KNm}, R_i = -R_j = 76.067 \text{ KN}
 \end{aligned}$$

No. : 9

$$\begin{aligned}
 l_x &= 10.0 \text{ m}, l_y = 2.0 \text{ m}, \omega_{\text{self}} = 12.81 \text{ KN/m (u.d.l.)} \\
 \omega_{\text{slab}} &= 31.415 \text{ KN/m \& } 5.235 \text{ KN/m (at node i\&j)} \\
 P &= 15.51 \text{ KN (Point load)} \\
 M_i &= 268.2472 \text{ KNm}, M_j = 202.1528 \text{ KNm} \\
 R_i &= 245.4643 \text{ KN}, R_j = 145.0957 \text{ KN} \\
 R_{ai} &= 55.6367 \text{ KN}, R_{aj} = 32.5935 \text{ KN}
 \end{aligned}$$

3.6 DYNAMIC ANALYSIS :: Force sets ::

Wind forces consideration :

For dynamic analysis wind and earthquake forces are considered. Here the wind forces comes out to be very small. Even if the section is treated as rectangular the total area comes out to be nearly 175 m^2 . And for a force of 104 kg/m^2 total force comes out to be 182 kN.

Earthquake forces consideration :

Place of construction - GORAKHPUR :: Earthquake zone IV

The seismic co-efficient method has been used to calculate dynamic forces for zone IV. During analysis the forces have been used as a lumped mass system.

$$\alpha_h = \beta I \alpha_o$$

$$\beta = 1.0, I = 1.5, \alpha_o = 0.05 \rightarrow \alpha_h = 0.075 = 7.5 \%$$

As the dynamic force due to wind is very less in comparison to that due to earthquake only seismic analysis has been carried out here.

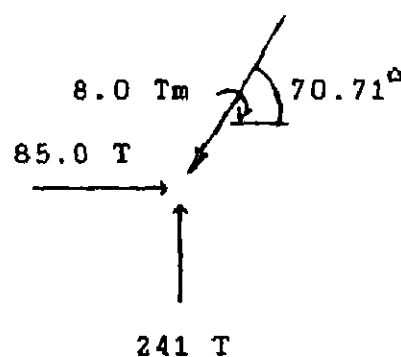
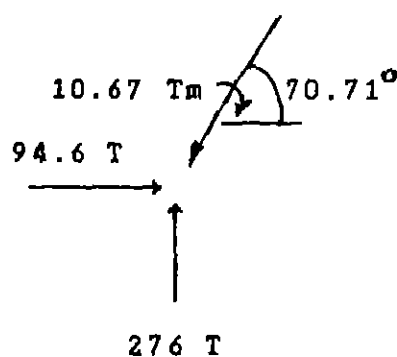
3.7 FOUNDATION DESIGN

PILE SPECIFICATIONS

Diameter : 500 mm. , Spacing : 2*Diameter

3.7.1 DESIGN OF PILE CAP BELOW COLUMNS

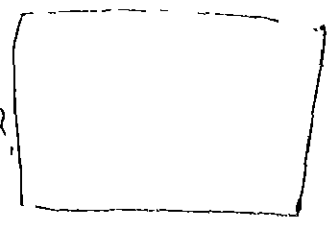
Loadings :



Shaft condition: FREE JOINT

FIXED JOINT

Load from column : 276 T
Self weight : 30 T
Total weight : 306 T

Is it pile capacity? 

No. of piles required = $306 / 60 = 5.1 \approx 6$

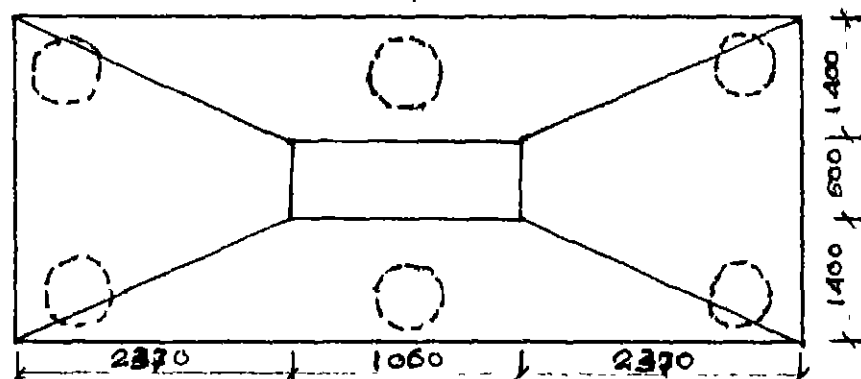


Fig.5 - PILE CAP BELOW COLUMN

Circular punching shear

Let depth of pile cap, $d = 25$ cm.

Diameter, $D = (0.5 + 0.15 + 0.125) = 0.75$ m = 75 cm.

Perimeter $= \pi * 0.75 = 2.356$ m = 235.6 cm.

Shear area $= 25 * 235.6 = 5890$ cm²

Shear stress $= 51000 / 5890 = 8.66$ kg/cm²

> 7.16 kg/cm²

Let depth of pile cap, $d = 30$ cm.

Diameter, $D = (0.5 + 0.15 + 0.15) = 0.8$ m = 80 cm.

Perimeter $= \pi * 0.8 = 2.51$ m = 251 cm.

Shear area $= 30 * 251 = 7530$ cm²

Shear stress $= 51000 / 7530 = 6.77$ kg/cm²

< 7.16 kg/cm²

So provide depth, $d = 30$ cm.

Total depth $= 30 + 30 = 60$ cm.

PUNCHING SHEAR UNDER COLUMN

Punching shear at (at $d/2$) :

Let depth, $d = 70$ cm

Perimeter $= (106 + 35 + 35) * 2 + (50 + 35 + 35) * 2 = 592$ cm

Shear area $= 592 * 70 = 41440$ cm²

Shear stress $= 306000 / 41440 = 7.38$ kg/cm²

Let depth, $d = 80$ cm

Perimeter $= (106 + 40 + 40) * 2 + (50 + 40 + 40) * 2 = 632$ cm

Shear area $= 632 * 80 = 50560$ cm²

Shear stress $= 306000 / 50560 = 6.052$ kg/cm²

< 7.16 kg/cm²

PUNCHING SHEAR FOR INDIVIDUAL PILE

Punching shear (at $d/2$) :

$$\begin{aligned}\text{Punching shear stress allowed} &= 0.16\sqrt{f_{ck}} \\ &= 7.16 \text{ Kg/cm}^2 \text{ for } M_{20} \\ \text{Punching shear force per pile} &= 306/6 \text{ T} \\ &= 51 \text{ T}\end{aligned}$$

Let depth of pile cap, $d = 50$ cm.

$$\begin{aligned}\text{Perimeter} &= (25+40+25)*4 = 360 \text{ cm.} \\ \text{Shear area} &= 360*50 = 18000 \text{ cm}^2 \\ \text{Shear stress,} &= 51000/18000 = 2.833 \text{ Kg/cm}^2 \\ &< 7.16 \text{ Kg/cm}^2\end{aligned}$$

Let depth of pile cap, $d = 35$ cm.

$$\begin{aligned}\text{Perimeter} &= (25+40+17.5)*4 = 330 \text{ cm} \\ \text{Shear area} &= 330*35 = 11550 \text{ cm}^2 \\ \text{Shear stress,} &= 51000/11550 = 4.416 \text{ Kg/cm}^2\end{aligned}$$

Let depth of pile cap, $d = 30$ cm.

$$\begin{aligned}\text{Perimeter} &= (25+40+15)*4 = 320 \text{ cm} \\ \text{Shear area} &= 320*30 = 9600 \text{ cm}^2 \\ \text{Shear stress,} &= 51000/9600 = 5.31 \text{ Kg/cm}^2 \\ &< 7.16 \text{ Kg/cm}^2\end{aligned}$$

Let depth of pile cap, $d = 25$ cm.

$$\begin{aligned}\text{Perimeter} &= (25+40+12.5)*4 = 310 \text{ cm} \\ \text{Shear area} &= 310*30 = 7750 \text{ cm}^2 \\ \text{Shear stress,} &= 51000/7750 = 6.58 \text{ Kg/cm}^2 \\ &< 7.16 \text{ Kg/cm}^2\end{aligned}$$

Let depth of pile cap, $d = 20$ cm.

$$\begin{aligned}\text{Perimeter} &= (25+40+10)*4 = 300 \text{ cm} \\ \text{Shear area} &= 300*20 = 6000 \text{ cm}^2 \\ \text{Shear stress,} &= 51000/6000 = 8.50 \text{ Kg/cm}^2 \\ &> 7.16 \text{ Kg/cm}^2\end{aligned}$$

Shear at d

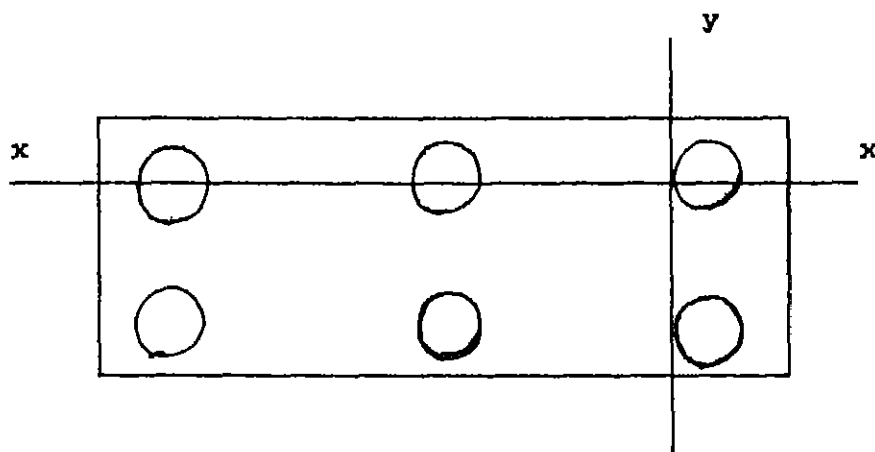


FIG-6 SECTION OF PILE CAP

Along Y axis

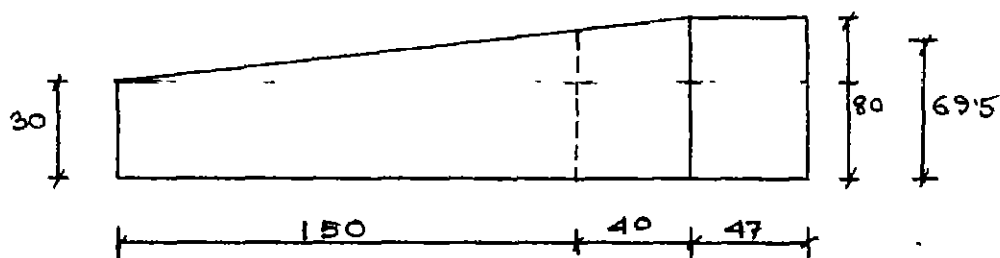


FIG-7A- Elevation of pile cap. along xx axis

$$\text{Shear area} = 69.5 \times 330 = 22935 \text{ cm}^2$$

$$\text{Shear force} = 2 \times 306 / 6 = 102 \text{ T}$$

$$\text{Shear stress} = 102000 / 22935 = 4.45 \text{ kg/cm}^2$$

$$< 7.16 \text{ kg/cm}^2$$

Along X axis

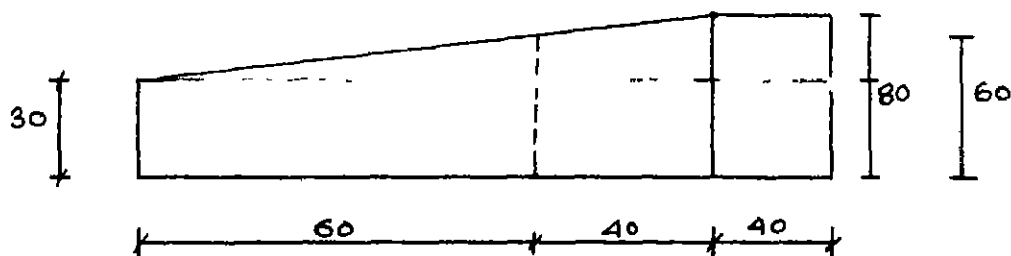


Fig-7B- Elevation along -yy axis

$$\text{Shear area} = 60 \times 580 = 34800 \text{ cm}^2$$

$$\text{Shear force} = 3 \times 306 / 6 = 153 \text{ T}$$

$$\text{Shear stress} = 153000 / 34800 = 4.40 \text{ kg/cm}^2$$

$$< 7.16 \text{ kg/cm}^2$$

DESIGN OF REINFORCEMENT

Along Y axis :

$$\text{Moment at face} = 102 \times 2.37$$

$$(\text{downward force}) = 241.74 \text{ T-m}$$

$$\text{Moment due to self weight} = 2.5 \times 3.3 \times \left(\frac{2.37^2 \times 1.1}{2} - 0.3 \times 1.5 \times 1.87 \right)$$

$$= 25.5 - 6.9 \text{ T-m}$$

$$= 18.5 \text{ T-m}$$

$$\text{Net moment} = 223.2 \text{ T-m}$$

$$\frac{M_u}{bd^2} = \frac{223.2 \times 1.5 \times 10^7}{3300 \times 800^2} = 1.585$$

$$\Rightarrow p_t = 0.494$$

$$\text{So, } A_{st} = 13041 \text{ mm}^2 \text{ provide } 25 \phi 125 \text{ c/c}$$

$$\Rightarrow A_{st} = 491 \times 28 = 13748 \text{ mm}^2 \text{ i.e. } 0.510 \%$$

$$\tau_c = 0.483 \text{ N/mm}^2$$

$$\text{Shear force} = 102 - 2.5 \times 3.3 \times (2.37 \times 1.1 - 0.3 - 1.5)$$

$$= 102 - 17.8 = 84.2 \text{ T}$$

$$\text{Shear stress} = \frac{84.2 \times 1.5 \times 10^4}{3300 \times 800} = 0.478 \text{ N/mm}^2$$

Along X axis :

$$\begin{aligned}\text{Moment at face} &= 153 \times 1.4 \\ (\text{downward force}) &= 213 \text{ T-m}\end{aligned}$$

$$\begin{aligned}\text{Moment due to self weight} &= 2.5 \times 5.8 \times \left(\frac{1.1 \times 1.4^2}{2} - 0.3 \times 1.0 \times 1.067 \right) \\ &= 13.3 \text{ T-m}\end{aligned}$$

$$\text{Net moment} = 200 \text{ T-m}$$

$$\frac{Mu}{bD^2} = \frac{200 \times 1.5 \times 10^7}{5800 \times 750^2} = 0.919$$

$$\Rightarrow p_t = 0.282\%$$

$$\text{So } A_{st} = 12180 \text{ mm}^2, \text{ provide } 20 \phi 125 \text{ c/c}$$

$$\Rightarrow A_{st} = 314 \times 48 = 15072 \text{ mm}^2 \text{ i.e. } 0.361\%$$

$$\tau_a = 0.413 \text{ N/mm}^2$$

$$\begin{aligned}\text{Shear force} &= 153 - 2.5 \times 5.8 \times (1.0 \times 1.1 - 0.3 \times 1.0) \\ &= 153 - 17.98 = 135 \text{ T}\end{aligned}$$

$$\begin{aligned}\text{Shear stress} &= \frac{135 \times 1.5 \times 10^4}{750 \times 5800} = 0.465 \text{ N/mm}^2\end{aligned}$$

Check for reactions

Moments generated :

From static analysis - 10.86 T-m

From dynamic analysis -	SN.	Mx	My	Mz
	1	0	0	79
	2	1.6	16	69
	3	2.8	28	40
	4	0	19	62
	5	3.2	33	0

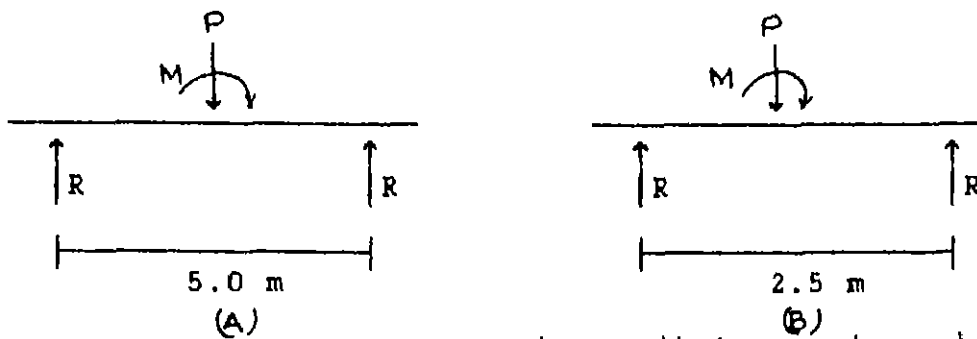


Fig-8- Reactions at piles below column

Reactions at the piles :

SN.	R1 (S+D)	R1 (Dyn.)	R2 (Dyn.)	R1+R2 (Max)
1	18.0	0	15.8	18.0
2	15.4	5.9	13.8	21.3
3	10.2	11.2	8.0	21.4
4	14.6	7.6	12.6	22.2
5	0	13.2	0	13.2

(All Values are in Tonnes)

$$P_{DL} + P_{LL} = 306/6 + 2.1 = 53.1 \text{ T}$$

$$P_{DL} + P_{LL} + P_{eq} = 51 + 22.2 = 73.2 \text{ T}$$

$$(P_{DL} + P_{LL} + P_{eq})/1.33 = 54.9 \text{ T} < 60 \text{ T} \text{ (Safe)}$$

3.7.2 DESIGN OF PILE CAP BELOW SHAFT

Loadings :

Shaft connection : FREE

FIXED

From top beam	12.06 T	15.78 T
From bottom beam	72.27 T	102.8 T
At midlanding	2.08 T =	4.49 T =
Total	86.41 T	122.07 T
Total for 12 parts	1037 T	1477 T
Self weight	123 T	123 T
Weight of piles	75 T	75 T
Total weight	1237 T	1675 T

Here the weight of the staircases and the domes etc. have not added considering they are small w.r.t. the total weight. For the design purpose the greater value of 1675 T was considered because for the design of pile caps under columns the greater value was taken into consideration. This was done to avoid settlement problem that may be generated if column and shaft have different stiffnesses.

No. of piles required : $1675/60 = 27.9 \approx 28$

The position of piles are shown in figure :

PUNCHING SHEAR FOR FOUNDATION BELOW SHAFT

Depth at $d/2 = 80$ cm.

Perimeter = $2 \times 240 = 1508$ cm.

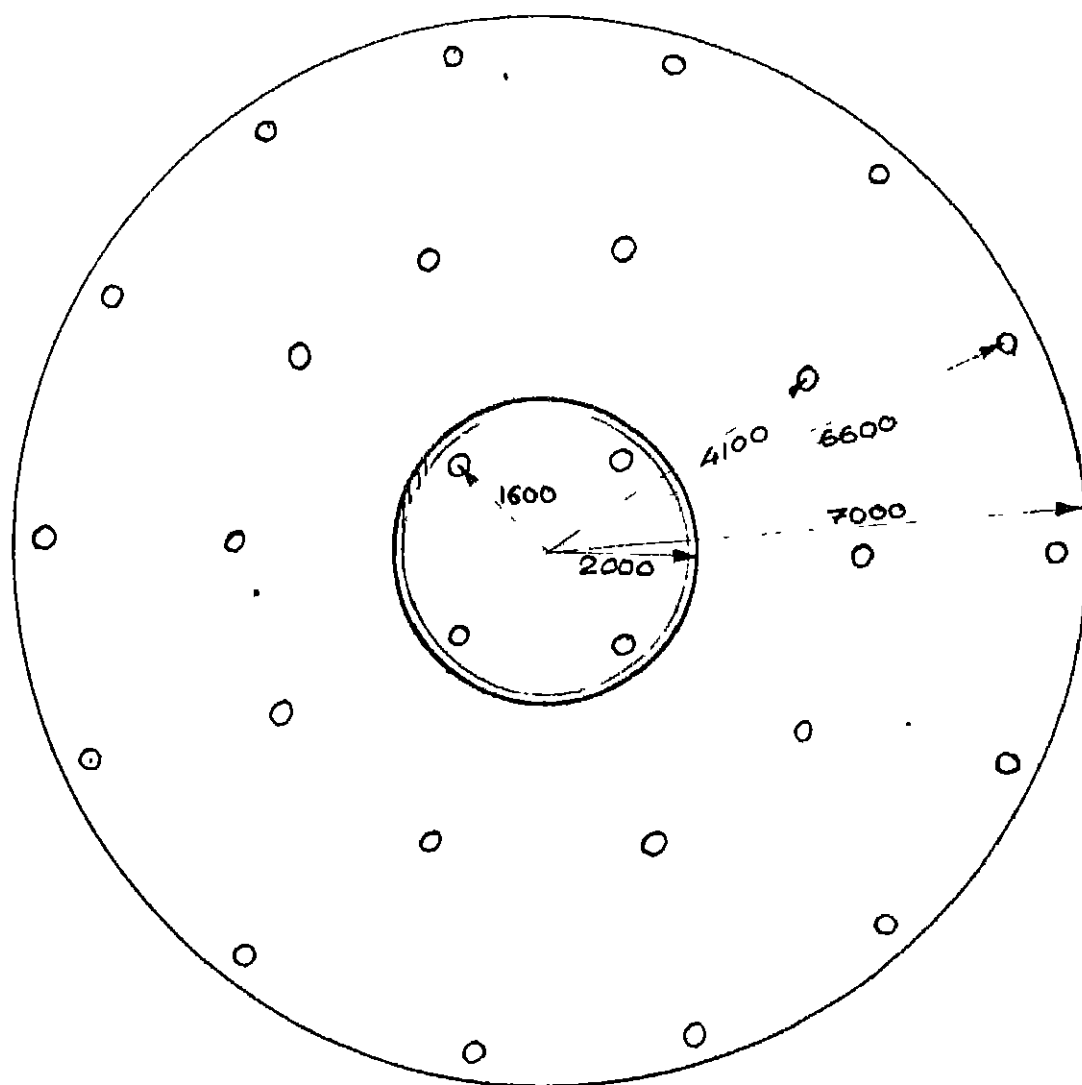
Shear area = $1508 \times 80 = 120640 \text{ cm}^2$

Total force = $840 - 4 \times 60 = 600$ T

Shear stress = $600 \times 103 / 120640 = 4.97 \text{ kg/cm}^2$

$< 7.16 \text{ kg/cm}^2$

So the section is safe against punching shear.



9

Fig.-9 Position of piles below shaft foundation

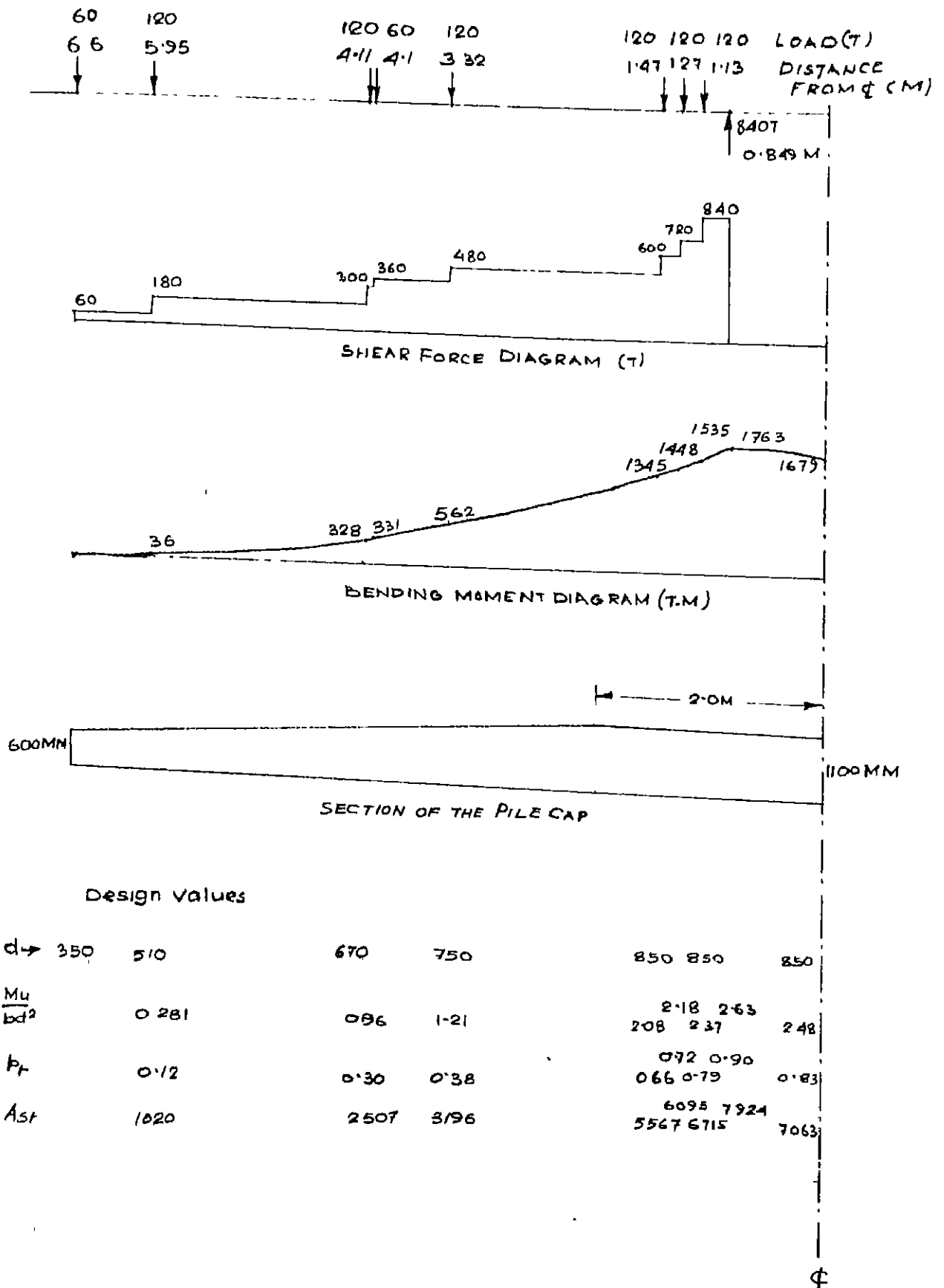


Fig.10- DIAGRAMS SHOWING FORCES
ON PILE CAP BELOW SHAFT

3.7.3 DESIGN OF CONNECTING BEAM AT FOUNDATION

Axial force = 96.4 T

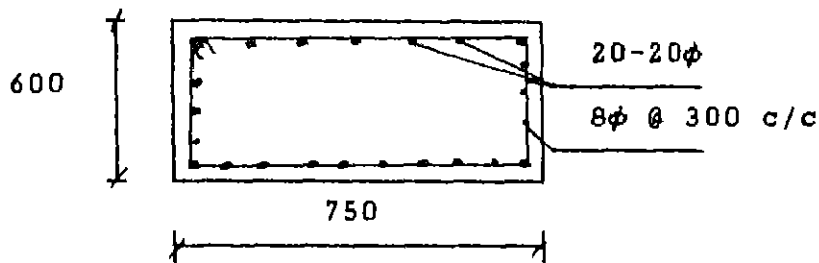


Fig 11 - Section of connecting beam at foundation

Area of steel = $314 \times 20 = 6280 \text{ mm}^2$

$$\begin{aligned}
 \text{Total force} &= 6280 \times 190 \times 10^{-4} + 750 \times 600 \times 1.7 \times 10^{-6} \\
 &= 120.22 \text{ T} \\
 &\approx 120 \text{ T}
 \end{aligned}$$

So the section is safe against tension.

3.8 DESIGN OF COLUMNS

Result from static analysis

Col. No.	End 1			End 2		
	P	M _x	V	P	M _x	V
1	291.5	10.67	+0.82	278.4	24.01	-5.41
2	171.5	26.92	+8.47	258.3	42.90	-3.74
3	38.61	123.2	+67.27	38.61	35.60	+5.20

Result from dynamic analysis

Col. No.	End 1			End 2		
	P	M _x	V	P	M _x	V
1	40.06	-79.38	-7.11	40.06	60.86	+7.11
2	45.99	89.21	16.63	45.99	109.90	-16.63
3	1.22	-33.56	-10.85	1.22	1.85	10.85

(P: AXIAL FORCE in T, M: MOMENT in T-m, V: SHEAR FORCE in T)

3.8.1 COLUMN NO. 1

Design for static case

End 1 : Axial force = 291.5 T , Moment = 10.67 T-m

End 2 : Axial force = 278.4 T , Moment = 24.01 T-m

$$l_x = l_y = \sqrt{(10.5^2 + 3.675^2)} = 11.125 \text{ m}$$

$$l/b = 11.125/0.5 = 22.250 > 12 \rightarrow \text{Slender column}$$

$$l/D = 11.125/1.0 = 11.125 < 12 \rightarrow \text{Short column}$$

Minimum eccentricity :

$$e_x = 11125/500 + 1000/30 = 56 \text{ mm}$$

$$e_y = 11125/500 + 500/30 = 40 \text{ mm}$$

Minimum moments :

$$\text{End 1 : } M_x = 0.056 * 291.5 = 16.32 \text{ T-m}$$

$$M_y = 0.040 * 291.5 = 11.66 \text{ T-m}$$

$$\text{End 2 : } M_x = 0.056 * 278.4 = 16.09 \text{ T-m}$$

$$M_y = 0.040 * 278.4 = 11.50 \text{ T-m}$$

Additional moments :

$$\text{End 1 : } M_{ay} = \frac{291.5 * 0.5}{2000} (11.125/0.5)^2$$

$$= 36.89 \text{ T-m}$$

$$\text{End 2 : } M_{ay} = \frac{278.4 * 0.5}{2000} (11.125/0.5)^2$$

$$= 35.24 \text{ T-m}$$

So design parameters for static analysis are ,

$$\text{End 1 : } P = 291.5 \text{ T, } M_x = 16.32 \text{ T-m, } M_y = 11.66 \text{ T-m}$$

$$M_{ay} = 36.89 \text{ T-m}$$

$$\text{End 2 : } P = 278.4 \text{ T, } M_x = 16.09 \text{ T-m, } M_y = 11.50 \text{ T-m}$$

$$M_{ay} = 35.24 \text{ T-m}$$

$$\text{Design moment, } M_x = 0.6 * 24.01 + 0.4 * 16.32 \text{ T-m}$$

$$= 20.96 \text{ T-m} \approx 21 \text{ T-m}$$

$$P_{ux} = 0.45 * 20 * 500000 + 0.75 * 415 * 14121$$

$$= 450.0 + 439.519 = 889.52 \text{ T}$$

$$P_u = 1.5 * 291.5 = 437.5 \text{ T}$$

Determination of P_{bal} :

$$d_f/D = 80/1000 = 0.08 \rightarrow k_1 = 0.207, k_2 = 0.328$$

$$p = 2.8\% \rightarrow p_b/f_{ck} bD = 0.253$$

$$P_{bal} = 252.9 \text{ T} \rightarrow K = 0.710$$

$$M_{uy} = 0.710 \cdot 36.89 + 11.66$$

$$= 37.85 \approx 38 \text{ T-m}$$

$$P_u / f_{ck} bD = \frac{437 \cdot 1.5 \cdot 10^4}{20 \cdot 500 \cdot 1000} = 0.432$$

$$M_{ux} / f_{ck} bD^2 = \frac{21 \cdot 1.5 \cdot 10^7}{20 \cdot 500 \cdot 1000^2} = 0.032$$

$$M_{uy} / f_{ck} Db^2 = \frac{38 \cdot 1.5 \cdot 10^7}{20 \cdot 1000 \cdot 500^2} = 0.114$$

$$\rightarrow p / f_{ck} = 0.134 \rightarrow p = 2.68\%$$

Let $p = 2.82\%$

$$P_u / P_{ux} = 0.492 \rightarrow \alpha_n = 1.486$$

$$M_{ux1} / f_{ck} bD^2 = 0.150 \rightarrow M_{ux1} = 150 \text{ T-m}$$

$$M_{uy1} / f_{ck} Db^2 = 0.135 \rightarrow M_{uy1} = 67.5 \text{ T-m}$$

$$\Rightarrow (M_{ux} / M_{ux1})^{\alpha_n} + (M_{uy} / M_{uy1})^{\alpha_n} = 0.078$$

So the section is safe at static condition

Results from (dynamic analysis)

Load combination?

End 1 : Axial force = 40.06 T , Moment = 79.38 T-m

End 2 : Axial force = 40.06 T , Moment = 60.86 T-m

Minimum moment $M_y = 0.040 \cdot 40.06 = 1.60 \text{ T-m}$

Additional moment: $M_{ay} = \frac{40.06 \cdot 0.5 \cdot (11.125 / 0.5)^2}{2000}$

$$= 4.96 \text{ T-m}$$

So design parameters for dynamic analysis are ,

$$P = 331.5 \text{ T}, M_x = 51.00 \text{ T-m}, M_y = 44.41 \text{ T-m}$$

*in static
dynamic?*

$$\begin{aligned}
 P_u &= 1.2 \times 331.5 = 397.8 \text{ T} \\
 M_{ux} &= 1.2 \times 51 = 61.2 \text{ T-m} \\
 M_{uy} &= 1.2 \times 44.41 = 53.29 \text{ T-m}
 \end{aligned}$$

$$P_u / P_{ux} = 0.447 \rightarrow \alpha_n = 1.412$$

$$M_{ux1} / f_{ck} b D^2 = 0.150 \rightarrow M_{ux1} = 150 \text{ T-m}$$

$$M_{uy1} / f_{ck} D b = 0.135 \rightarrow M_{uy1} = 67.5 \text{ T-m}$$

$$\Rightarrow (M_{ux} / M_{ux1})^{\alpha_n} + (M_{uy} / M_{uy1})^{\alpha_n} = 0.933$$

So the section is safe at dynamic condition

3.8.2 COLUMN NO. 2

Design for static case

End 1 : Axial force = 271.5 T , Moment = 26.92 T-m

End 2 : Axial force = 258.3 T , Moment = 42.90 T-m

$$l_x = l_y = \sqrt{(10.8^2 + 3.78^2)} = 11.442 \text{ m}$$

$$l/b = 11.442 / 0.5 = 22.884 > 12 \rightarrow \text{Slender column}$$

$$l/D = 11.442 / 1.0 = 11.442 < 12 \rightarrow \text{Short column}$$

Minimum eccentricity :

$$e_x = 11442 / 500 + 1000 / 30 = 56.5 \text{ mm}$$

$$e_y = 11442 / 500 + 500 / 30 = 40 \text{ mm}$$

Minimum moments :

$$\text{End 1 : } M_x = 0.056 \times 271.5 = 15.34 \text{ T-m}$$

$$M_y = 0.040 \times 271.5 = 10.86 \text{ T-m}$$

$$\text{End 2 : } M_x = 0.056 \times 258.3 = 14.59 \text{ T-m}$$

$$M_y = 0.040 \times 258.3 = 10.33 \text{ T-m}$$

Additional moments :

104195

$$\text{End 1 : } M_{ay} = \frac{271.5 \times 0.5}{2000} \times (11.442/5.0)^2$$

$$= 35.54 \text{ T-m}$$

$$\text{End 2 : } M_{ay} = \frac{258.3 \times 0.5}{2000} \times (11.442/5.0)^2$$

$$= 33.82 \text{ T-m}$$

So design parameters for static analysis are ,

$$\text{End 1 : } P = 271.5 \text{ T}, M_x = 26.92 \text{ T-m}, M_y = 10.86 \text{ T-m}$$

$$M_{ay} = 35.54 \text{ T-m}$$

$$\text{End 2 : } P = 258.3 \text{ T}, M_x = 42.90 \text{ T-m}, M_y = 10.33 \text{ T-m}$$

$$M_{ay} = 33.82 \text{ T-m}$$

$$\text{Design moment, } M_x = 0.6 \times 42.90 + 0.4 \times 26.92 \text{ T-m}$$

$$= 36.51 \text{ T-m} \approx 36.5 \text{ T-m}$$

$$P_{ux} = 0.45 \times 20 \times 5000000 + 0.75 \times 415 \times 14121$$

$$= 450.0 + 439.519 = 889.52 \text{ T}$$

$$P_u = 1.5 \times 271.5 = 407.25 \text{ T}$$

Determination of P_{bal} :

$$d_1/D = 80/1000 = 0.08 \rightarrow k_1 = 0.207, k_2 = 0.328$$

$$p = 2.8\% \rightarrow p_b/f_{ck} bD = 0.2529$$

$$P_{bal} = 264.4 \text{ T} \rightarrow K = 0.765$$

$$M_{uy} = 0.710 \times 35.54 + 10.86$$

$$= 36.09 \approx 36 \text{ T-m}$$

$$P_u/f_{ck} bD = \frac{407 \times 1.5 \times 10^4}{20 \times 500 \times 1000} = 0.41$$

$$M_{ux}/f_{ck} bD^2 = \frac{36.5 \times 1.5 \times 10^7}{20 \times 500 \times 1000^2} = 0.055$$

$$M_{uy}/f_{ck} Db^2 = \frac{36 \times 1.5 \times 10^7}{20 \times 1000 \times 500^2} = 0.108$$

$$\begin{aligned} \rightarrow p/f_{ak} &= 0.120 \quad \rightarrow p = 2.40\% \\ p &= 2.8\% \end{aligned}$$

$$P_u/P_{uz} = 0.458 \quad \rightarrow \alpha_n = 1.430$$

$$\begin{aligned} M_{ux1}/f_{ak} b D^2 &= 0.150 \quad \rightarrow M_{ux1} = 150 \quad \text{T-m} \\ M_{uy1}/f_{ak} D b^2 &= 0.135 \quad \rightarrow M_{uy1} = 67.5 \quad \text{T-m} \end{aligned}$$

$$\Rightarrow (M_{ux}/M_{ux1})^{\alpha_n} + (M_{uy}/M_{uy1})^{\alpha_n} = 0.963$$

So the section is safe at static condition

Results from dynamic analysis

End 1 : Axial force = 45.99 T , Moment = 89.26 T-m

End 2 : Axial force = 45.99 T , Moment = 109.9 T-m

$$\text{Minimum moment } M_y = 0.040 \times 45.99 = 1.84 \text{ T-m}$$

$$\begin{aligned} \text{Additional moment: } M_{ay} &= \frac{45.99 \times 0.5}{2000} \times (11.442/0.50)^2 \\ &= 6.02 \text{ T-m} \end{aligned}$$

So design parameters for dynamic analysis are ,

$$P = 317.5 \text{ T}, M_x = 66.50 \text{ T-m}, M_y = 45.80 \text{ T-m}$$

$$P_u = 1.2 \times 331.5 = 397.8 \text{ T}$$

$$M_{ux} = 1.2 \times 66.25 = 79.50 \text{ T-m}$$

$$M_{uy} = 1.2 \times 37.92 = 45.52 \text{ T-m}$$

$$P_u/P_{uz} = 0.428 \quad \rightarrow \alpha_n = 1.381$$

$$M_{ux1}/f_{ak} b D^2 = 0.150 \quad \rightarrow M_{ux1} = 150 \quad \text{T-m}$$

$$M_{uy1}/f_{ak} D b^2 = 0.135 \quad \rightarrow M_{uy1} = 67.5 \quad \text{T-m}$$

$$\Rightarrow (M_{ux}/M_{uxd})^{\alpha_n} + (M_{uy}/M_{uyd})^{\alpha_n} = 0.996$$

So the section is safe at dynamic condition

3.8.3 COLUMN NO. 3

Design for static case

End 1 : Axial force = 38.61 T , Moment = 123.2 T-m

End 2 : Axial force = 38.61 T , Moment = 35.60 T-m

$$l_x = l_y = 5.65 \text{ m}$$

$$l/b = 5.65/0.5 = 11.3 < 12 \rightarrow \text{Short column}$$

$$l/D = 5.65/1.0 = 5.65 < 12 \rightarrow \text{Short column}$$

END 1:

$$\frac{P_{u_w}}{f_{ck}bD} = \frac{38.61 \times 1.5 \times 10^4}{20 \times 500 \times 1000} = 0.06$$

$$\frac{M_{ux}/f_{ck}bD^2}{20 \times 500 \times 1000^2} = \frac{123.2 \times 1.5 \times 10^7}{20 \times 500 \times 1000^2} = 0.185$$

$$\rightarrow p/f_{ck} = 0.15 \rightarrow p = 3.0\%$$

END 2:

$$\frac{P_u}{f_{ck}bD} = \frac{38.61 \times 1.5 \times 10^4}{20 \times 500 \times 600} = 0.06$$

$$\frac{M_{ux}}{f_{ck}bD^2} = \frac{35.60 \times 1.5 \times 10^7}{20 \times 500 \times 600^2} = 0.148 \quad (\text{Taking } d=600 \text{ mm})$$

$$\rightarrow p/f_{ck} = 0.11 \rightarrow p = 2.2\%$$

Results from dynamic analysis

End 1 : Axial force = 1.22 T , Moment = 35.56 T-m

End 2 : Axial force = 1.22 T , Moment = 1.85 T-m

Minimum moment $M_y = 0.040 \times 40.06 = 1.60 \text{ T-m}$

$$P_u = 1.2 \times 40.83 = 49 \text{ T}$$

The section is safe for dynamic loading because the static loading is much higher than dynamic loading.

3.9 DESIGN OF SHAFT

$$\text{Total load} = 1450 + 10 \% = 1595 \text{ T}$$

$$\begin{aligned} \text{Moment} &= 0.075 * (91.82 * 24.125 + 110.3 * 14.25 + 20.13 * 12.0) \\ &= 303 \text{ Tm} \end{aligned}$$

$$\text{Let thickness of the shaft} = 0.125 \text{ m.}$$

$$\text{Outer diameter} = 4.0 \text{ m, Inner diameter} = 3.75 \text{ m}$$

$$\text{Mean diameter} = 3.875 \text{ m, Area} = 1.5217 \text{ m}^2$$

$$\text{1st moment of area, I} = 2.85915 \text{ m}^4$$

$$e = M/w = 0.209 \text{ m, } e/r = 0.108, \tau_{abc} = 7.0 \text{ N/mm}^2$$

$$\Rightarrow \frac{M}{\tau_{cb} \cdot Z_{cb}} = \frac{3030}{0.125 * 1.9735 * 7000} = 0.922$$

$$\begin{aligned} P_u &= 0.4 * f_{ak} * A_a + 0.67 * f_y * A_{sc} \\ &= 1.21756 * 107 + 3.82 * 106 \\ &= 1217 + 382 = 1599 \text{ T} \end{aligned}$$

$$P_{alv} = 1066 \text{ T}$$

So the section is not suitable

$$\text{Let thickness of the shaft} = 0.140 \text{ m.}$$

$$\text{Outer diameter} = 4.0 \text{ m, Inner diameter} = 3.72 \text{ m}$$

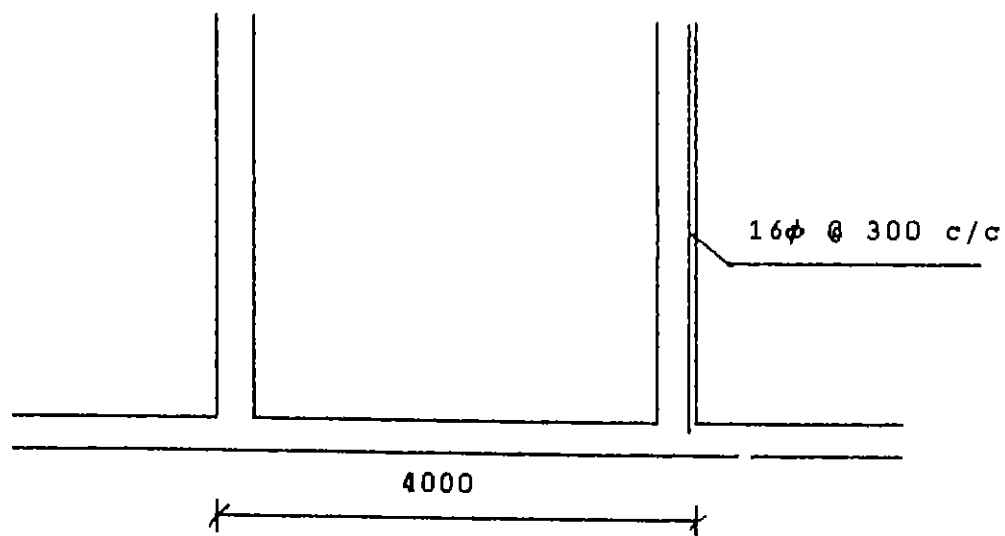
$$\text{Mean diameter} = 3.80 \text{ m, Area} = 1.6977 \text{ m}^2$$

$$\text{1st moment of area, I} = 3.16607 \text{ m}^4$$

$$\begin{aligned} P_u &= 0.4 * f_{ak} * A_a + 0.67 * f_y * A_{sc} \\ &= 1358 + 1416 = 2774 \text{ T} \end{aligned}$$

$$P_a = 1849 \text{ T}$$

$$\begin{aligned} M &= Z * \tau_{abc} \\ &= 1.583 * 7 = 1108.125 \text{ Tm} > 303 \text{ Tm} \end{aligned}$$



Section of shaft

Fig.-12

3.10 DESIGN OF BEAMS

3.10.1 BOTTOM RADIAL BEAM : RB1

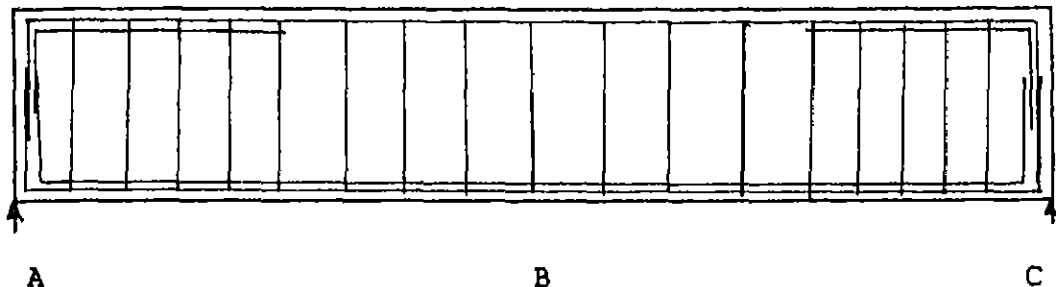


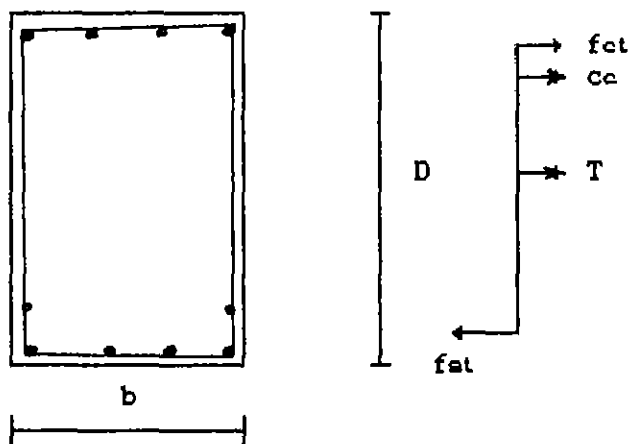
Fig-13A - SECTION OF A TYPICAL BEAM

M : 737.4 KNm	1840 KNm	1222 KNm
V : 1288 KN	Negligible	979 KN
T : 776 KN	776 KN	776 KN

Designatend A

$$M_u = 111 \text{ Tm}, V_u = 194 \text{ T}, T_u = 117 \text{ T}$$

$$A_{st(min)} = 0.20\% \implies A_{st} = 1500 \text{ cm}^2 \implies 2-32\phi$$



$$x_u = 280 \text{ mm}$$

Fig 13B - SECTION OF A TYPICAL BEAM

$$\begin{aligned} \implies c_c &= 0.36 \times 280 \times 600 \times 20 \times 10^{-4} = 121 \text{ T at 118 mm from top} \\ F_{ac} &= 1608 \times 354 \times 10^{-4} = 57 \text{ T} \\ F_{st} &= 1608 \times 361 \times 10^{-4} = 58 \text{ T} \end{aligned}$$

$$T_u = 121 + 57 - 58 = 120 \text{ T}$$

$$M_u = 121 \times 1.414 + 57 \times 1.218 - 120 \times 0.609 = 134 \text{ Tm}$$

$$M_u = 120 \times 0.609 + 58 \times 1.218 - 121 \times 0.077 = 134 \text{ Tm}$$

So the section is safe in flexure.

$$A_{st} = 1608 \text{ mm}^2 \Rightarrow 100A_{st}/bd = 0.212 \Rightarrow \tau_c = 0.252$$

$$\tau_{ve} = 194 \times 10^4 / (600 \times 1259) = 2.56 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$$

$$S_v = 0.87 \times 415 / (2.31 \times 600) = 0.26 A_{sv}$$

So the section is safe in shear

Considering only flexure and shear :

$$M_u/bd^2 = 111 \times 10^7 / (600 \times 1200^2) = 1.284$$

$$\Rightarrow p_t = 0.392 \rightarrow A_{st} = 2822 \text{ mm}^2$$

Provide 4 - 25 ϕ + 4 - 20 ϕ (at top)

$$p_t = 0.43\% , \tau_v = 2.613 , S_v = 0.277 A_{sv}$$

Provide 12 ϕ 4 legged @ 100 c/c.

Designatend C

$$M_u = 184 \text{ Tm}, V_u = 147 \text{ T}, T_u = 117 \text{ T}$$

$$A_{st(min)} = 0.20\% \Rightarrow A_{st} = 1500 \text{ mm}^2 \Rightarrow 2-32\phi$$

$$\text{Let } x_u = 340 \text{ mm}$$

$$\Rightarrow c_d = 0.36 \times 340 \times 600 \times 20 \times 10^{-4} = 147 \text{ T at 142 mm from top}$$

$$F_{sc} = 2594 \times 354.8 \times 10^{-4} = 92 \text{ T}$$

$$F_{st} = 3217 \times 361 \times 10^{-4} = 116 \text{ T}$$

$$T_u = 147 + 92 - 116 = 123 \text{ T}$$

$$M_u = 147 \times 1.116 + 92 \times 1.218 - 123 \times 0.609 = 201 \text{ Tm}$$

$$M_u = 123 \times 0.609 + 116 \times 1.218 - 147 \times 0.102 = 201 \text{ Tm}$$

So the section is safe in flexure.

$$A_{st} = 3217 \text{ mm}^2 \Rightarrow 100A_{st}/bd = 0.425 \Rightarrow \rho = 0.442$$

$$\tau_{ve} = 147 \times 10^4 / (600 \times 1259) = 1.95 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$$

$$S_v = 0.87 \times 415 / (1.502 \times 600) = 0.40 A_{sv}$$

Considering only flexure and shear :

$$M_u / bd^2 = 184 \times 10^7 / (600 \times 1200^2) = 2.13$$

$$\Rightarrow p_t = 0.693 \quad \text{--->} \quad A_{st} = 4990 \text{ mm}^2$$

Provide 8 - 25 ϕ + 4 - 20 ϕ (at top)

$$p_t = 0.698 \% , \tau_v = 1.980 , S_v = 0.418 A_{sv}$$

Provide 10 ϕ 4 legged @ 100 c/c.

Designatend B

$$M_u = 276 \text{ Tm}, \quad T_u = 117 \text{ T}$$

$$A_{st(min)} = 0.20\% \Rightarrow A_{st} = 1500 \text{ cm}^2 \Rightarrow 2-32 \phi$$

$$b_f = 7/6 + 0.6 + 0.25 \times 6 = 3.267 \text{ m}$$

$$\text{Let } x_u = 105 \text{ mm}$$

$$\Rightarrow c_c = 0.36 \times 105 \times 3266 \times 20 \times 10^{-4} = 247 \text{ T at 44 mm from top}$$

$$F_{sc} = 1608 \times 324.8 \times 10^{-4} = 52 \text{ T}$$

$$F_{st} = (3217 + 1608) \times 361 \times 10^{-4} = 174 \text{ T}$$

$$T_u = 247 + 52 - 174 = 125 \text{ T}$$

$$M_u = 247 \times 1.215 + 52 \times 1.214 - 125 \times 0.609 - 58 \times 0.609 = 284 \text{ Tm}$$

$$M_u = 58 \times 1.154 + 116 \times 1.218 + 0.609 \times 125 - 247 \times 0.003 = 284 \text{ Tm}$$

So the section is safe in flexure.

Considering only flexure and shear :

$$M_u / bd^2 = 276 \times 10^7 / (3267 \times 1200^2) = 0.586$$

$$\Rightarrow p_t = 0.172 \quad \text{--->} \quad A_{st} = 6743 \text{ mm}^2$$

Provide 12 - 25 ϕ + 4 - 20 ϕ (at top)

$$p_t = 0.180 \% \rightarrow S_v = 0.317 A_{sv}$$

Provide 12 ϕ 2 legged @ 125 c/c.

3.10.2 TOP RADIAL BEAM : RB2

Place	A	B	C
M :	282 KNm	147 KNm	120 KNm
V :	270 KN	Negligible	120 KN
T :	163 KN	118.5 KN	74 KN

Designatend A

$$M_u = 42.3 \text{ Tm}, V_u = 40.5 \text{ T}, T_u = 24.5 \text{ T}$$

$$\text{Let } x_u = 225 \text{ mm}$$

$$\Rightarrow c_c = 0.36 \times 225 \times 300 \times 20 \times 10^{-4} = 47.5 \text{ T at 92 mm from top}$$

$$F_{sc} = 402 \times 355 \times 10^{-4} = 14.27 \text{ T}$$

$$F_{st} = 2413 \times 361 \times 10^{-4} = 87.11 \text{ T}$$

$$T_u = 87.11 - 14.27 - 47.5 = 25.34 \text{ T}$$

$$M_u = 47.5 \times 0.5596 + 14.27 \times 0.628 + 25.35 \times 0.302 = 43.2 \text{ T-m}$$

$$M_u = 87.11 \times 0.628 - 25.35 \times 0.302 - 47.5 \times 0.0068 = 43.8 \text{ T-m}$$

So the section is safe in flexure.

$$A_{st} = 2413 \text{ mm}^2 \Rightarrow 100A_{st}/bd = 1.233 \Rightarrow \rho = 0.62$$

$$\tau_{va} = 40.5 \times 10^4 / (300 \times 652) = 2.07 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$$

$$S_v = 0.87 \times 415 / (1.45 \times 300) = 0.83 A_{sv}$$

So the section is safe in shear

Provide 4 - 25 ϕ + 4 - 20 ϕ (at top)

$$\rho_t = 0.43 \% , \tau_v = 2.613 , S_v = 0.277 A_{sv}$$

Provide 12 ϕ 4 legged @ 100 c/c.

Designatend C

$$M_u = 18 \text{ Tm}, V_u = 18 \text{ T}, T_u = 11.1 \text{ T}$$

Let $x_u = 75 \text{ mm}$

$$\begin{aligned} \Rightarrow c_c &= 0.36 \times 75 \times 300 \times 20 \times 10^{-4} = 16 \text{ T at } 31.5 \text{ mm from top} \\ F_{ac} &= 226 \times 342 \times 10^{-4} = 7.73 \text{ T-m} \\ F_{at} &= 982 \times 361 \times 10^{-4} = 35.45 \text{ T-m} \end{aligned}$$

$$T_u = 35.45 - 16 - 7.73 = 11.52 \text{ T}$$

$$M_u = 7.73 \times 0.6415 + 16.2 \times 0.631 + 11.52 \times 0.302 = 18.66 \text{ T-m}$$

$$M_u = 35.45 \times 0.6415 - 16.2 \times 0.0105 - 11.52 \times 0.302 = 19.09 \text{ T-m}$$

So the section is safe in flexure.

$$A_{at} = 982 \text{ mm}^2 \Rightarrow 100A_{at}/bd = 0.494 \Rightarrow \tau_c = 0.475$$

$$\tau_{ve} = 18 \times 10^4 / (300 \times 662) = 0.906 \text{ N/mm}^2 < 2.8 \text{ N/mm}^2$$

$$S_v = 0.87 \times 415 / (0.431 \times 600) = 2.79 A_{av}$$

So the section is safe in shear

Provide 8 - 25ϕ + 4 - 20ϕ (at top)

$$p_t = 0.698 \% , \tau_v = 1.980 , S_v = 0.418 A_{av}$$

Provide 10 ϕ 4 legged @ 100 c/c.

Designatend B

$$M_u = 276 \text{ Tm}, T_u = 117 \text{ T}$$

$$A_{at(min)} = 0.20\% \Rightarrow A_{at} = 1500 \text{ cm}^2 \Rightarrow 2-320/$$

Let $x_u = 115 \text{ mm}$

$$\begin{aligned} \Rightarrow c_c &= 0.36 \times 115 \times 300 \times 20 \times 10^{-4} = 24.8 \text{ T at } 48 \text{ mm from top} \\ F_{ac} &= 226 \times 342 \times 10^{-4} = 7.73 \text{ T} \\ F_{at} &= 1432 \times 361 \times 10^{-4} = 51.7 \text{ T} \end{aligned}$$

$$T_u = 51.69 - 7.73 - 24.8 = 25.61 \text{ T}$$

$$M_u = 7.73 \times 0.631 + 24.84 \times 0.603 + 19 \times 0.302 = 25.61 \text{ T-m}$$

$$M_u = 51.69 \times 0.631 - 24.84 \times 0.0273 - 19 \times 0.302 = 26.02 \text{ T-m}$$

So the section is safe in flexure.

Provide 12 - 25ϕ + 4 - 20ϕ (at top)

$$p_t = 0.180 \% \rightarrow S_v = 0.317 A_{av}$$

Provide 12 ϕ 2 legged @ 125 c/c.

3.10.3 BOTTOM CIRCULAR BEAM : CB1

Explain.

The circular beams were idealised as straight beams for analysis purpose.

Axial force = 1282 KN, Shear force = 309 KN, Moment = 177 KNm

As a curved member ,

$$\omega = 9.0 + 18.375 + 32.5 + 46 \times 1.15 = 113.235 \text{ KN/m}$$

(for slab, wall, self weight & water)

$$M = M_o \cos \theta - \omega r^2 (1 - \cos \theta)$$

$$T = M_o \sin \theta - \omega r^2 (-\sin \theta)$$

$$M_o = K_4 \omega r^2, r = 11.6 \text{ m}, K_4 = 0.04$$

$$\Rightarrow M_o = 609 \text{ KNm}$$

$$\rightarrow \theta = 0^\circ : M = M_o = 609 \text{ KNm}, T = 0$$

$$\rightarrow \theta = 15^\circ : M = 69.5 \text{ KNm}, T = 112.33 \text{ KNm}$$

So design parameters :

At support - $M_f = 177 \text{ KNm}$, $M_t = 112 \text{ KNm}$, $A = 1282 \text{ KN}$, $V = 309 \text{ KN}$

At midspan - $M_f = 364 \text{ KNm}$, $M_t = 0$, A (axial force) = 1282 KN

DESIGN AT SUPPORT

$$M_u = 1.5 [18 + 11.2 (1 + 1300/750) / 1.7] = 54 \text{ Tm}$$

$$V_u = 1.5 [30.9 + 1.6 (11.2 / 0.75)] = 82 \text{ T}, T_u = 1.5 \times 128.2 = 192.3 \text{ T}$$

$$\text{Let } x_u = 455 \text{ mm}$$

$$\Rightarrow C_u = 0.36 \times 455 \times 750 \times 20 \times 10^{-4} = 245.7 \text{ T at } 191 \text{ mm from top}$$

$$F_{sc} = 982 \times 355 \times 10^{-4} = 43.86 \text{ T}$$

$$F_{st} = 2413 \times 361 \times 10^{-4} = 87.11 \text{ T}$$

$$\Rightarrow T_u = 245.7 + 43.86 - 87.11 = 193.45 \text{ T}$$

$$M_u = 34.86 \times 1.2215 + 245.7 \times 1.0679 - 193.45 \times 0.609 = 187.15 \text{ Tm}$$

$$100 A_{st} / bd = 0.255 \rightarrow \tau_u = 0.362 \rightarrow \tau_{vu} = 0.868$$

$$\begin{aligned}
 A_{sv} &= (27 \times 10^7 / 700 + 30.9 \times 1.5 \times 10^4) S_v / 1221.5 \times 361 \\
 &= (0.875 + 0.420) S_v \\
 \Rightarrow S_v &= 0.772 A_{sv} \quad \text{or} \\
 S_v &= 361 / 0.506 \times 750 = 0.951 A_{sv}
 \end{aligned}$$

Design only as a flextural member.

$$\begin{aligned}
 M_u / bd^2 &= 0.4608 \rightarrow p_t = 0.143 \\
 &\text{Provide } 0.20 \% \text{ i.e., } 1875 \text{ mm}^2
 \end{aligned}$$

$$\begin{aligned}
 \tau_o &= 0.25 \%, \tau_v = 0.874 \text{ N/mm}^2, S_v = 0.771 A_{sv} \\
 &\text{Provide } 12\phi \text{ 2 legged @ } 150 \text{ c/c}
 \end{aligned}$$

DESIGN AT MIDSPAN

$$\begin{aligned}
 M_u &= 55 \text{ Tm}, T_u = 193.2 \text{ T} \\
 b_f &= 0.7 \times 0.24 / 12 + 0.75 + 6 \times 0.25 = 2.65 \text{ m} > 1.8 \text{ m}
 \end{aligned}$$

Here the design parameters are same as those of the earlier ones.

3.10.4 DESIGN OF BOTTOM CIRCULAR BEAMS : CB2

$$\begin{aligned}
 l &= 4.922 \text{ m}, w_{self} = 0.4 \times 1.025 \times 2.5 = 1.025 \text{ KN/m} \\
 w_{slab} &= 1.2 \times 0.25 \times 2.5 = 0.75 \text{ T/m}, w_{water} = 1.2 \times 4.6 = 5.52 \text{ T/m} \\
 w_{total} &= 7.295 \text{ KN/m} \approx 7.3 \text{ T/m}
 \end{aligned}$$

$$\begin{aligned}
 \text{Fixed end moment} &= w l^2 / 12 = 14.74 \text{ Tm} \approx 15 \text{ Tm} \\
 \text{Midspan moment} &= 15 / 2 = 7.5 \text{ Tm} \\
 \text{Moment at face} &= 15 - (\omega l - \omega x) x / 2 = 9.93 \text{ Tm} \approx 10 \text{ Tm}
 \end{aligned}$$

$$\begin{aligned}
 \text{Shear force} &= \omega l / 2 = 17.96 \text{ T} \approx 18 \text{ T} \\
 \text{Shear at } d/2 &= 18 - \omega d / 2 = 15.81 \text{ T} \approx 16 \text{ T}
 \end{aligned}$$

DESIGNAT SUPPORT

$M_u/bd^2 = 0.24$ for $d = 400$ mm & 0.32 for $d = 300$ mm
 $\Rightarrow p_t = 0.09\%$ \rightarrow Provide minimum reinforcement (0.20%)

$\tau_{ve} = 0.64$ N/mm², $\tau_o = 0.36$ N/mm², $S_v = 4.30$ A_{sv}
 \rightarrow Provide minimum transverse reinforcement.

DESIGNAT MIDSPAN

Here the design parameters are same as those at support, so the design will be same as the design at support. this is also because minimum reinforcements have been provided at the section.

3.10.5 DESIGN OF BOTTOM CIRCULAR BEAMS :CB3:

$l = 3.456$ m & loading is same as loading of beam CB2

So the design will be same as beam CB2

3.10.6 DESIGN OF TOP CIRCULAR BEAMS :CB6:

Here also minimum reinforcements should be provided because of small span and smaller loading.

3.10.7 DESIGN OF BOTTOM CIRCULAR RING :CB4:

This circular ring is to be designed for tension only.
 Let force from each beam is T .

Then force in the ring = $T(1+2\sin 30+2\sin 60)$

So tension = 3.73×76.59 T = 285.83 T

$A_{st} = 285.83 \times 1.5 \times 104 / 0.87 \times 415 = 11875$ mm²

Provide 15 - 32 ϕ

3.10.8 DESIGN OF TOP CIRCULAR RING :CB7:

This circular ring also is to be designed for tension only.

Let force from each beam is T.

Then force in the ring = $T(1+2\sin 30+2\sin 60) = 3.73 T$

So tension = $3.73 \times 16 T = 59.7$

$$A_{st} = 59.7 \times 1.5 \times 10^4 / 0.87 \times 415 = 11875 \text{ mm}^2$$

Provide 8 - 20 ϕ

3.11 DESIGN OF SLAB

Materials : Concrete - M_{20} , Steel - Fe 415

3.11.1 BOTTOM SLAB :

The bottom slab is resting on four circular beams and two radial beams. So it has to be designed in three parts. For the design the annular slab are idealized as rectangular slabs where the radial distance and the middle arc length are considered as the two sides of the rectangular slab. The radial distances between the intermediate circular beams are adjusted in such a way so as to give nearly equal depth of slab from primary calculations (by trial and error method).

Explain

For preliminary calculations depth of slab is calculated from the equation ,

$$D = \sqrt[3]{(M \cdot 6 / 1700)} \quad \text{for } M_{20} \text{ concrete}$$

where , D = depth in m.

M = moment in KNm/m

Part A : $l_y = 5.498 \text{ m}$; $l_x = 2.2 \text{ m}$; $l_y / l_x = 2.499 > 2.0$
So it has to be designed as one way slab.

$$M_x = w l^2 / 12.0 = 53.5 \cdot 2.2^2 / 12.0 = 21.699 \text{ KNm/m}$$

$$\text{So } D = \sqrt[3]{(21.699 \cdot 6 / 1700)} = 0.277 \text{ m.}$$

Part B : $l_y = 4.489 \text{ m}$; $l_x = 2.8 \text{ m}$; $l_y / l_x = 1.496 < 2.0$
So it has to be designed as two way slab.

$$-x = 0.053 \Rightarrow -M_x = 0.053 \cdot 2.8^2 = 22.23 \text{ KNm/m}$$

$$\text{So } D = \sqrt[3]{(22.23 \cdot 6 / 1700)} = 0.280 \text{ m.}$$

Part C : $l_x = 2.251 \text{ m}$; $l_y = 4.6 \text{ m}$; $l_y / l_x = 2.043 > 2.0$
So it has to be designed as one way slab.

$$M_x = w l^2 / 12.0 = 53.5 \cdot 2.251^2 / 12.0 = 22.59 \text{ KNm/m}$$

$$\text{So } D = \sqrt[3]{(22.59 \cdot 6 / 1700)} = 0.283 \text{ m.}$$

Determination of reinforcements :

According to the INDIAN STANDARD CODE OF PRACTICE the minimum reinforcement is 0.3% for slab of depth 100 mm or less and 0.2% for slab of depth 450 mm or more. For slabs of intermediate depth the percentage of steel should be varied linealy.

Depth of neutral axis,

$$x = (0.5bD^2 + (m-1)pbDd) / (bD + (m-1)pbD)$$

$$I_{NA} = bx^3/3 + b(D-x)^3/3 + (m-1)pbD(d-x)^2$$

$$M_R = f_{tb} * I_{NA} / (D-x)$$

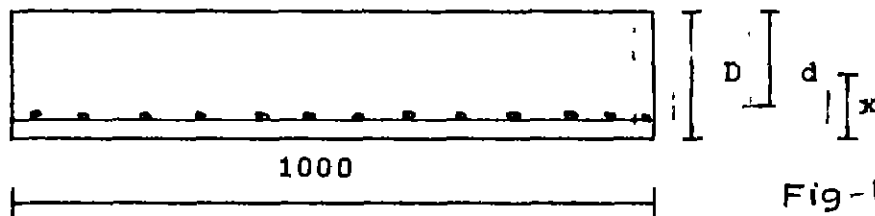


Fig-14A SECTION OF
LIQUID RETAINING
SLAB

Here let the percentage of steel is 0.3%.

So $p = 0.003$, $D = 275$ mm, $d = 245$ mm

$m = 280 / (7 * 3) = 13$, $f_{tb} = 1.7$ N/mm² for M_{20}

$x = 141.235$ mm, $I_{NA} = 1.844 * 10^6$ mm⁴

$$\Rightarrow M_R = 23.438 \text{ KNm/m}$$

So reinforcement of 0.3% is provided everywhere.

3.11.2 TOP SLAB :

The bottom slab is resting on three circular beams and two radial beams . So it has to be designed in two parts.

Part A : $l_y = 6.2$ m ; $l_x = 2.67$ m ; $l_y / l_x = 2.32 > 2.0$

So it has to be designed as one way slab.

$$M_x = wl^2 / 12.0 = 7.0 * 2.67^2 / 12.0 = 4.15 \text{ KNm/m}$$

$$\text{So } D = \sqrt{(4.15 * 6 / 1700)} = 0.277 \text{ m.}$$

Part B : $l_y = 5.34 \text{ m}$; $l_x = 4.0 \text{ m}$; $l_y/l_x = 1.335 < 2.0$

So it has to be designed as two way slab.

$$-x = 0.049 \Rightarrow -M_x = 0.049 \times 4.0^2 = 5.488 \text{ KNm/m}$$

$$\text{So } D = \sqrt{(5.488 \times 6 / 1700)} = 0.280 \text{ m.}$$

So let the design depth is 150 mm.

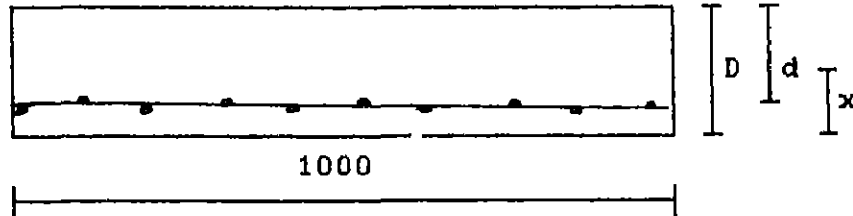


FIG-145 SECTION OF LIQUID RETAINING SLAB

Here let the percentage of steel is 0.3%.

$$\text{So } p = 0.003, D = 275 \text{ mm}, d = 245 \text{ mm}$$

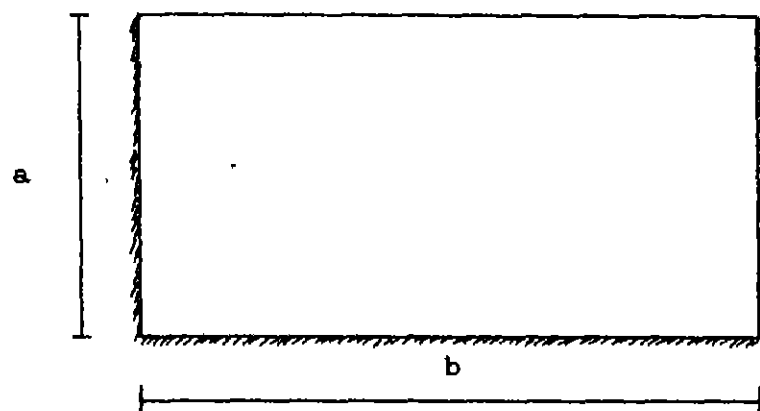
$$m = 280 / (7 \times 3) = 13, f_{tb} = 1.7 \text{ N/mm}^2$$

$$x = 76.7375 \text{ mm}, I_{NA} = 2.943 \times 10^8 \text{ mm}^4$$

$$\Rightarrow M_x = 6.283 \text{ KNm/m}$$

So reinforcement of 0.3% is provided everywhere.

3.12 DESIGN OF TANK WALL



$$a = 4.6$$

$$b = 5.862$$

$$\omega = 10 \text{ kN/m}^2$$

$$b/a = 1.274$$

Fig 15A - SECTION OF
WALL FOR
MOMENT
: COEFFICIENTS

Moment co-efficients for a slab

TABLE-1: MOMENT CO-EFFICIENTS FOR TANK WALL

b/a	x/a	y=0		y=b/4		b/2	
		α_x	α_y	α_x	α_y	α_x	α_y
1.50	0.0	0.0	0.021	0.001	0.005	0.0	-0.040
	0.25	0.008	0.020	0.004	0.007	-0.009	-0.044
	0.50	0.016	0.016	0.010	0.008	-0.008	-0.042
	0.75	-0.003	-0.006	0.003	-0.004	-0.005	-0.026
	1.00	-0.060	-0.012	-0.041	-0.008	0.0	0.0
1.25	0.0	0.0	0.015	0.0	0.003	0.0	-0.029
	0.25	0.005	0.015	0.002	0.005	-0.007	-0.034
	0.50	0.014	0.015	0.008	0.007	-0.007	-0.037
	0.75	0.006	0.007	0.005	0.005	-0.005	-0.024
	1.00	-0.047	-0.009	-0.031	-0.006	0.0	0.0
1.27	0.0	0.0	0.016	0.0	0.003	0.0	-0.030
	0.25	0.005	0.016	0.002	0.005	-0.007	-0.034
	0.50	0.014	0.015	0.008	0.007	-0.007	-0.038
	0.75	0.005	0.006	0.005	0.004	-0.005	-0.024
	1.00	-0.048	-0.009	-0.032	-0.006	0.0	0.0

Moments at the tank wall

$$M_x = \alpha_x \omega a^3, \quad M_y = \alpha_y \omega a^3$$

b/a	x/a	y=0		y=b/4		b/2	
		M_x	M_y	M_x	M_y	M_x	M_y
1.27	0.0	0.0	15.18	0.0	3.11	0.0	-29.20
	0.25	5.16	15.08	2.14	5.06	-7.01	-34.07
	0.5	13.82	14.70	7.98	6.91	-6.91	-36.50
	0.75	5.06	5.55	4.67	3.99	-4.87	-23.55
	1.0	-47.01	-9.05	-31.15	-6.03	0.0	0.0

TABLE 2— MOMENTS IN TANK WALL

Calculation of moment carrying capacity of section

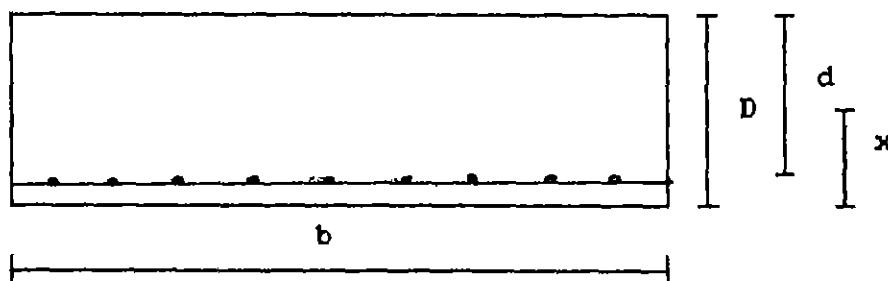


FIG-15B - SECTION OF WALL FOR MOMENTS

DEPTH(D)	DEPTH(d)	STEEL	N.A. (x)	I_{NA}	M_R
(mm)	(mm)	%	(mm)	10^8 mm^4	KNm/m
375	344	0.6	198.0	50.11	48.14
300	269	0.5	156.7	24.90	29.55
318	288	0.6	168.0	30.50	34.41

TABLE 3 - MOMENT RESISTANCE CAPACITY OF SLAB

3.13 DESIGN OF MIDLANDING PLATE :PLT:

This plate has to be designed as compression member.

The force at a section is $3.73 T$ where T is the force transferred from the node at column junction.

$$T = 128 + 2 \times 78.33 \times \sin 75$$

So compressive force at plate section is,

$$= 1042.4 \times 104 / 250 \times 17000 = 2.45 \text{ N/mm}^2 \text{ (safe)}$$

So provide $8 \phi @ 200 \text{ c/c}$.

3.14 DESIGN OF STAIRCASE :STC:

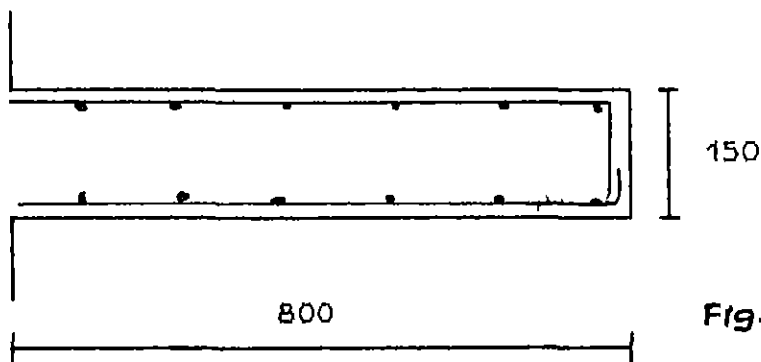


Fig.-16 SECTION OF STAIRCASE

$$\text{Live load} = 300 \text{ kg/m}^2 = 0.3 \text{ T/m}^2$$

$$\text{Dead load} = 2.5 \times 0.15 = 0.375 \text{ T/m}^2$$

$$\text{Railing} = 25 \text{ kg/m} = 0.025 \text{ T/m}$$

$$\text{Total moment} = 0.065 \text{ Tm} \Rightarrow M_u / bd^2 = 0.866$$

$$p_t = 0.264 \%, A_{st} = 39.6 \text{ mm}^2$$

Provide 2 - 12ϕ

3.15 REVISED WEIGHT OF THE WATER TANK

MATERIAL : 1 ::CL1::

$\text{Length} = \sqrt{(10.52+3.6752)} = 11.125 \text{ m}$
 $\text{Total length} = 11.125*12 = 133.50 \text{ m}$
 $\text{Weight} = 0.5*1.0*133.5*25 = 1668.75 \text{ KN}$
 $\text{Cummulative weight} = 1668.75 \text{ KN}$

MATERIAL : 2 ::CL2::

$\text{Length} = \sqrt{(10.82+3.782)} = 11.442 \text{ m}$
 $\text{Total length} = 11.442*12 = 137.31 \text{ m}$
 $\text{Weight} = 0.5*1.0*137.3*25 = 1716.36 \text{ KN}$
 $\text{Cummulative weight} = 3385.11 \text{ KN}$

MATERIAL : 3 ::CL3::

$\text{Length} = 5.65 \text{ m}$
 $\text{Total length} = 5.65*12 = 67.8 \text{ m}$
 $\text{Weight} = 0.5*(1.0+0.7)*67.8*25/2 = 720.34 \text{ KN}$
 $\text{Cummulative weight} = 4105.98 \text{ KN}$

MATERIAL : 4 ::PLT::

$\text{Area} = \pi(8.52-22)^2 - 1-12*0.5*0.5 = 204.41 \text{ m}^2$
 $\text{Weight} = 201.41*0.3*25 = 1578.10 \text{ KN}$
 $\text{Cummulative weight} = 5684.09 \text{ KN}$

MATERIAL : 5 ::CB1::

$\text{Length} = 2*\pi*12.0 = 75.398 \text{ m}$
 $\text{Weight} = 0.750*1.025*25*75.398 = 1449.06 \text{ KN}$
 $\text{Cummulative weight} = 7133.15 \text{ KN}$

MATERIAL : 6 ::RB1::

$\text{Length} = 12.375-2.0-0.75-0.3 = 9.325 \text{ m}$
 $\text{Total length} = 12*9.375 = 111.9 \text{ m}$
 $\text{Weight} = 111.9*0.6*1.025*25 = 1720.46 \text{ KN}$
 $\text{Cummulative weight} = 8853.61 \text{ KN}$

MATERIAL : 7 ::CB5::

$\text{Length} = 2*\pi*12.05 = 75.71 \text{ m}$
 $\text{Weight} = 75.71*0.35*0.5*25 = 331.24 \text{ KN}$
 $\text{Cummulative weight} = 9184.85 \text{ KN}$

MATERIAL : 8 ::RB2::

$\text{Length} = \sqrt{(9.72+22)} = 9.90 \text{ m}$
 $\text{Total length} = 9.90*12 = 118.85 \text{ m}$
 $\text{Weight} = 118.85*0.3*0.55*25 = 490.25 \text{ KN}$
 $\text{Cummulative weight} = 9675.10 \text{ KN}$

MATERIAL : 9 ::CB2::

Length = $2 * 9.4 - 12 * 0.5 = 53.06 \text{ m}$
 Weight = $53.06 * 0.3 * 1.025 * 25 = 454.04 \text{ KN}$
 Cumulative weight = 10129.14 KN

MATERIAL : 10 ::CB3::

Length = $2 * 6.6 - 12 * 0.5 = 35.47 \text{ m}$
 Weight = $35.47 * 0.3 * 1.025 * 25 = 272.67 \text{ KN}$
 Cumulative weight = 10401.81 KN

MATERIAL : 11 ::CB4::

Length = $2 * 2.15 = 13.50 \text{ m}$
 Weight = $13.50 * 0.3 * 1.025 * 25 = 103.85 \text{ KN}$
 Cumulative weight = 10505.66 KN

MATERIAL : 12 ::CB6::

Length = $2 * 8.35 - 12 * 0.3 = 48.86 \text{ m}$
 Weight = $48.86 * 0.3 * 0.35 * 25 = 128.27 \text{ KN}$
 Cumulative weight = 10633.93 KN

MATERIAL : 13 ::CB7::

Length = $2 * 2.15 = 13.50 \text{ m}$
 Weight = $13.50 * 0.3 * 0.35 * 25 = 35.46 \text{ KN}$
 Cumulative weight = 10669.39 KN

MATERIAL : 14 ::TWL::

Length = $2 * 12.2 - 12 * 0.5 = 70.65 \text{ m}$
 Weight = $70.65 * 4.9 * 0.3375 * 25 = 2912.14 \text{ KN}$
 Cumulative weight = 13590.52 KN

MATERIAL : 15 ::SL1::

Area = $*(12.3752 - 22) = 468.54 \text{ m}^2$
 Weight = $468.54 * 0.275 * 25 = 3221.20 \text{ KN}$
 Cumulative weight = 16811.73 KN

MATERIAL : 16 ::SL2::

Area = $*(13.3752 - 22) = 549.44 \text{ m}^2$
 Weight = $549.44 * 0.15 * 25 = 2060.38 \text{ KN}$
 Cumulative weight = 18872.11 KN

MATERIAL : 17 ::SFT::

Area = $*(22 - 1.862) = 1.6977 \text{ m}^2$
 Weight = $29.5 * 1.6977 * 25 = 1252.07 \text{ KN}$
 Cumulative weight = 20124.18 KN

ESTIMATION OF REINFORCEMENT

ITEM	TYPE	SHAPE	LENGTH	DISTANCE	SPACING	NO. DIAMETER	TOTAL LENGTH
1. CB1	TOP LONG.	CIRCULAR	62.83			6	376.99
	BOT. LONG.	CIRCULAR	62.83			6	376.99
	LAP	STRAIGHT	1.04			6X5X2	62.40
	FACE	CIRCULAR	62.83			4	251.32
	LAP	STRAIGHT	0.624			4X5	12.48
	TRANSVERSE	SQUARE	3.844	5.720	175	12X34	1568.35
2. CB2	TRANSVERSE	SQUARE	3.344	5.720	175	12X34	1364.35
	TOP LONG.	CIRCULAR	50.27			3	150.80
	BOT. LONG.	CIRCULAR	50.27			4	201.06
	LAP	STRAIGHT	0.832			7X4	23.30
	FACE	CIRCULAR	50.27			4	201.06
	LAP	STRAIGHT	0.624			4X4	9.98
3. CB3	TRANVERSE	SQUARE	1.472	4.420	250	19X12	335.62
	TOP LONG.	CIRCULAR	30.16			3	90.48
	BOT. LONG.	CIRCULAR	30.16			4	120.64
	LAP	STRAIGHT	0.832			7X2	11.65
	FACE	CIRCULAR	30.16			4	120.64
	LAP	STRAIGHT	0.624			4X2	4.99
4. CB4	TRANVERSE	SQUARE	1.472	3.955	250	17X12	300.29
	LONGTUDINAL	CIRCULAR	13.509			32	432.28
	TRANVERSE	SQUARE	1.472	0.615	300	3X12	52.99
	TOP LONG.	CIRCULAR	75.71			3	227.14
	LAP	STRAIGHT	1.300			3X5	19.50
	BOT. LONG.	CIRCULAR	75.71			2	151.24
5. CB5	LAP	STRAIGHT	1.300			2X5	13.00
	FACE	CIRCULAR	75.71			2	151.24
	TRANVERSE	SQUARE	1.040			2X5	10.40
	TOP LONG.	CIRCULAR	75.71			3	227.14
	LAP	STRAIGHT	1.300			3X5	19.50
	BOT. LONG.	CIRCULAR	75.71			2	151.24

6. CB6	TOP	LONG.	CIRCULAR	52.46			2	25	104.93
	LAP		STRAIGHT	1.300			2X4	25	10.40
	BOT.	LONG.	CIRCULAR	52.46			2	25	104.93
	LAP		STRAIGHT	1.300			2X4	25	10.40
	TRANSVERSE		SQUARE	1.544	3.870	175	24X12	8	444.67
7. CB7	LONGITUDINAL		CIRCULAR	13.509			8	20	108.07
	TRANSVERSE		SQUARE	1.344	0.615	300	3X12	8	48.38
8. RB1	TOP	LONG.	STRAIGHT	11.525			4X12	25	553.20
	TOP	LONG.	STRAIGHT	4.275			4X12	12	205.20
	TOP	LONG.	STRAIGHT	4.275			4X12	25	205.20
	TOP	LONG.	STRAIGHT	2.860			4X12	20	137.80
	BOT.	LONG.	STRAIGHT	11.525			6X12	25	829.80
	BOT.	LONG.	STRAIGHT	7.775			6X12	25	559.80
	BOT.	LONG.	STRAIGHT	6.375			4X12	20	306.00
	FACE		STRAIGHT	10.375			4X12	12	498.00
	TRANSVERSE		SQUARE	3.616	3.200	100	33X12	12	1431.96
	TRANSVERSE		SQUARE	3.416	3.200	100	33X12	12	1352.76
	TRANSVERSE		SQUARE	3.616	3.225	125	27X12	12	1171.56
	TRANSVERSE		SQUARE	3.416	3.225	125	27X12	12	1106.76
	TRANSVERSE		SQUARE	3.580	3.200	100	33X12	10	1417.68
	TRANSVERSE		SQUARE	3.380	3.200	100	33X12	10	1138.48
9. RB2	TOP	LONG.	STRAIGHT	12.275			4X12	20	590.16
	TOP	LONG.	STRAIGHT	4.375			3X12	25	157.50
	BOT.	LONG.	STRAIGHT	12.295			2X12	20	295.08
	BOT.	LONG.	STRAIGHT	8.000			2X12	25	192.00
	TRANSVERSE		SQUARE	1.816	3.275	150	23X12	12	501.22
10. F1	TRANSVERSE		SQUARE	1.780	6.530	250	27X12	10	576.72
	TOP	LONG.	STRAIGHT	6.150	3.300	125	28X12	25	2066.40
	TOP	LONG.	STRAIGHT	3.650	5.800	125	48X12	20	2102.40
11. F2	LONGITUDINAL		STRAIGHT	4.675			20X12	20	1122.00
	TRANSVERSE		SQUARE	2.844	2.475	300	10X12	8	341.28

12.F3	LONGITUDINAL STRAIGHT	4.000	4.000	100	41X2	32	328.00
	LONGITUDINAL STRAIGHT	VARIABLE	14.000	100	71X2	25	1463.21
13.SFT	MAIN STRAIGHT	31.000	11.690	100	118	16	3658.00
	DISTRIBUTION CIRCULAR	12.068	20.500	200	138	8	1665.38
	DISTRIBUTION CIRCULAR	12.150	8.450	150	57	10	692.55
14.CL1	MAIN STRAIGHT	31.891			16X12	32	6123.07
CL2	TRANSVERSE SQUARE	2.944	26.417	250	107X12	8	3780.10
CL3	TRANSVERSE SQUARE	2.344	26.417	500	107X12X2	8	6619.39
	TRANSVERSE SQUARE	1.644	26.417	500	107X12X2	8	4221.79
15.SL1	MAIN : B1 STRAIGHT	2.090	5.810	150	39X12	8	978.12
	MAIN : B2 STRAIGHT	3.060	3.870	150	27X12	8	991.44
	MAIN : B4 STRAIGHT	2.252	0.626	150	5X12	8	151.20
	MAIN : B5 STRAIGHT	3.380	3.300	150	22X12	8	892.32
	MAIN : B6 STRAIGHT	1.849	5.900	150	40X12	8	887.50
	MAIN : A1 STRAIGHT	4.050	5.150	150	36X12	10	1749.60
	MAIN : A2 STRAIGHT	5.970	5.150	150	36X12	10	2579.04
	MAIN : A4 STRAIGHT	6.400	2.250	150	16X12	10	1288.80
	MAIN : A5 STRAIGHT	7.390	2.250	150	16X12	10	1418.88
	MAIN : A7 STRAIGHT	4.489	3.300	150	22X12X2	10	2370.19
	MAIN : A8 STRAIGHT	2.549	5.900	150	22X12X2	10	2447.04
	DIST.: B1 STRAIGHT	5.919	0.990	150	8X12	6	710.28
	DIST.: B2 STRAIGHT	3.872	2.760	150	20X12	6	929.29
	DIST.: B3 STRAIGHT	1.168	1.770	150	12X12	6	168.14
	DIST.: B4 STRAIGHT	3.300	2.880	150	20X12	6	792.00
	DIST.: B5 STRAIGHT	5.900	1.349	150	10X12	6	708.00

16.SL2	MAIN : B1	STRAIGHT	1.208	5.460	120	46X12	10	666.54
	MAIN : B2	STRAIGHT	1.628	4.707	120	40X12	10	781.20
	MAIN : B3	STRAIGHT	2.460	3.034	120	26X12	10	767.52
	MAIN : B4	STRAIGHT	2.090	1.042	120	9X12	10	225.72
	MAIN : B5	STRAIGHT	3.613	1.525	120	13X12	10	563.56
	MAIN : B6	STRAIGHT	2.917	2.900	120	25X12	10	875.24
	MAIN : B7	STRAIGHT	1.598	4.300	120	36X12	10	690.34
	MAIN : A1	STRAIGHT	2.750	5.198	120	44X12	12	1452.00
	MAIN : A2	STRAIGHT	3.070	5.189	120	44X12	12	1620.76
	MAIN : A3	STRAIGHT	4.940	4.029	120	34X12	12	2015.52
	MAIN : A4	STRAIGHT	4.408	4.029	120	34X12	12	1789.26
	MAIN : A5	STRAIGHT	5.050	1.830	120	16X12	12	969.26
	MAIN : A6	STRAIGHT	6.370	1.830	120	16X12	12	1223.04
	MAIN : A7	STRAIGHT	7.796	1.525	120	13X12X2	12	2432.25
	MAIN : A8	STRAIGHT	6.289	2.900	120	25X12X2	12	3773.25
	MAIN : A9	STRAIGHT	3.429	4.300	120	36X12X2	12	2962.40
	DIST.: B1	STRAIGHT	5.460	0.478	150	4X12	8	262.08
	DIST.: B2	STRAIGHT	4.709	1.328	150	9X12	8	508.57
	DIST.: B3	STRAIGHT	3.032	2.160	150	15X12	8	548.12
	DIST.: B4	STRAIGHT	1.042	1.770	150	12X12	8	150.05
	DIST.: B5	STRAIGHT	1.525	3.130	150	21X12	8	384.30
	DIST.: B6	STRAIGHT	2.900	2.417	150	17X12	8	591.60
	DIST.: B7	STRAIGHT	4.300	1.098	150	8X12	8	412.80

17.ULL	MAIN : A	STRAIGHT	1.920	5.930	100	60X12	8	1382.40
	MAIN : B <td>STRAIGHT <td>2.670 <td>5.930 <td>120 <td>60X12 <td>16</td> <td>1922.40</td> </td></td></td></td></td>	STRAIGHT <td>2.670 <td>5.930 <td>120 <td>60X12 <td>16</td> <td>1922.40</td> </td></td></td></td>	2.670 <td>5.930 <td>120 <td>60X12 <td>16</td> <td>1922.40</td> </td></td></td>	5.930 <td>120 <td>60X12 <td>16</td> <td>1922.40</td> </td></td>	120 <td>60X12 <td>16</td> <td>1922.40</td> </td>	60X12 <td>16</td> <td>1922.40</td>	16	1922.40
	MAIN : C <td>STRAIGHT <td>6.900 <td>5.930 <td>100 <td>60X12X2</td> <td>12</td> <td>9936.00</td> </td></td></td></td>	STRAIGHT <td>6.900 <td>5.930 <td>100 <td>60X12X2</td> <td>12</td> <td>9936.00</td> </td></td></td>	6.900 <td>5.930 <td>100 <td>60X12X2</td> <td>12</td> <td>9936.00</td> </td></td>	5.930 <td>100 <td>60X12X2</td> <td>12</td> <td>9936.00</td> </td>	100 <td>60X12X2</td> <td>12</td> <td>9936.00</td>	60X12X2	12	9936.00
	MAIN : D <td>STRAIGHT <td>4.058 <td>4.900</td> <td>140 <td>36X12</td> <td>16</td> <td>1753.06</td> </td></td></td>	STRAIGHT <td>4.058 <td>4.900</td> <td>140 <td>36X12</td> <td>16</td> <td>1753.06</td> </td></td>	4.058 <td>4.900</td> <td>140 <td>36X12</td> <td>16</td> <td>1753.06</td> </td>	4.900	140 <td>36X12</td> <td>16</td> <td>1753.06</td>	36X12	16	1753.06
	MAIN : E <td>STRAIGHT <td>8.459 <td>4.900</td> <td>100 <td>50X12X2</td> <td>12</td> <td>10150.80</td> </td></td></td>	STRAIGHT <td>8.459 <td>4.900</td> <td>100 <td>50X12X2</td> <td>12</td> <td>10150.80</td> </td></td>	8.459 <td>4.900</td> <td>100 <td>50X12X2</td> <td>12</td> <td>10150.80</td> </td>	4.900	100 <td>50X12X2</td> <td>12</td> <td>10150.80</td>	50X12X2	12	10150.80
	DIST. : A <td>CIRCULAR <td>77.150</td> <td>1.470</td> <td>150</td> <td>10</td> <td>10</td> <td>771.50</td> </td>	CIRCULAR <td>77.150</td> <td>1.470</td> <td>150</td> <td>10</td> <td>10</td> <td>771.50</td>	77.150	1.470	150	10	10	771.50
	DIST. : B <td>CIRCULAR <td>77.150</td> <td>1.470</td> <td>150</td> <td>10</td> <td>10</td> <td>771.50</td> </td>	CIRCULAR <td>77.150</td> <td>1.470</td> <td>150</td> <td>10</td> <td>10</td> <td>771.50</td>	77.150	1.470	150	10	10	771.50
	DIST. : C <td>CIRCULAR <td>77.150</td> <td>4.900</td> <td>150</td> <td>34</td> <td>10</td> <td>2623.10</td> </td>	CIRCULAR <td>77.150</td> <td>4.900</td> <td>150</td> <td>34</td> <td>10</td> <td>2623.10</td>	77.150	4.900	150	34	10	2623.10
	DIST. : D <td>STRAIGHT <td>4.900</td> <td>3.558</td> <td>150</td> <td>24X12</td> <td>10</td> <td>1411.20</td> </td>	STRAIGHT <td>4.900</td> <td>3.558</td> <td>150</td> <td>24X12</td> <td>10</td> <td>1411.20</td>	4.900	3.558	150	24X12	10	1411.20
	DIST. : E <td>STRAIGHT <td>4.900</td> <td>5.930</td> <td>150</td> <td>40X12</td> <td>10</td> <td>2352.00</td> </td>	STRAIGHT <td>4.900</td> <td>5.930</td> <td>150</td> <td>40X12</td> <td>10</td> <td>2352.00</td>	4.900	5.930	150	40X12	10	2352.00

18.PLT TOTAL	STRAIGHT	VARIABLE	ALTHROUGH	200	86	8	1150.56
--------------	----------	----------	-----------	-----	----	---	---------

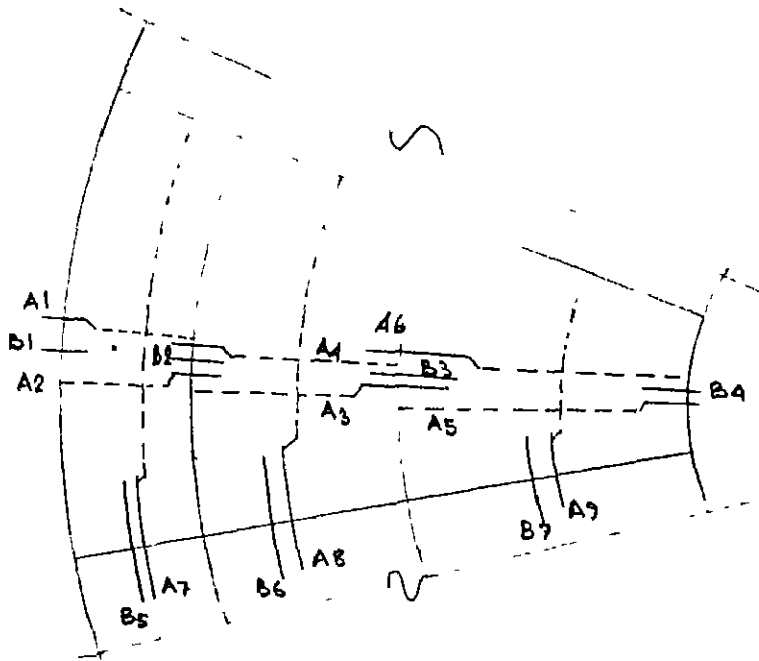


Fig-17 DIAGRAM FOR DETAILING OF REINFORCEMENT
IN SLAB

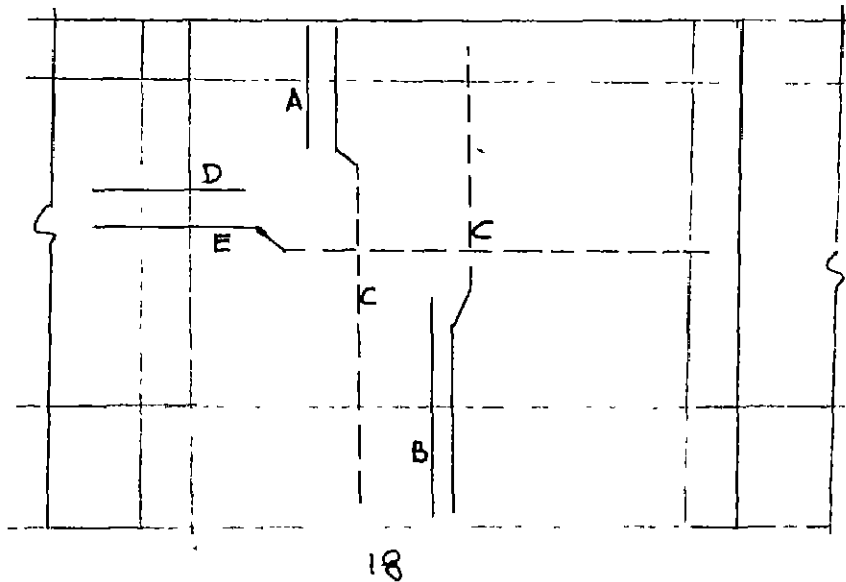
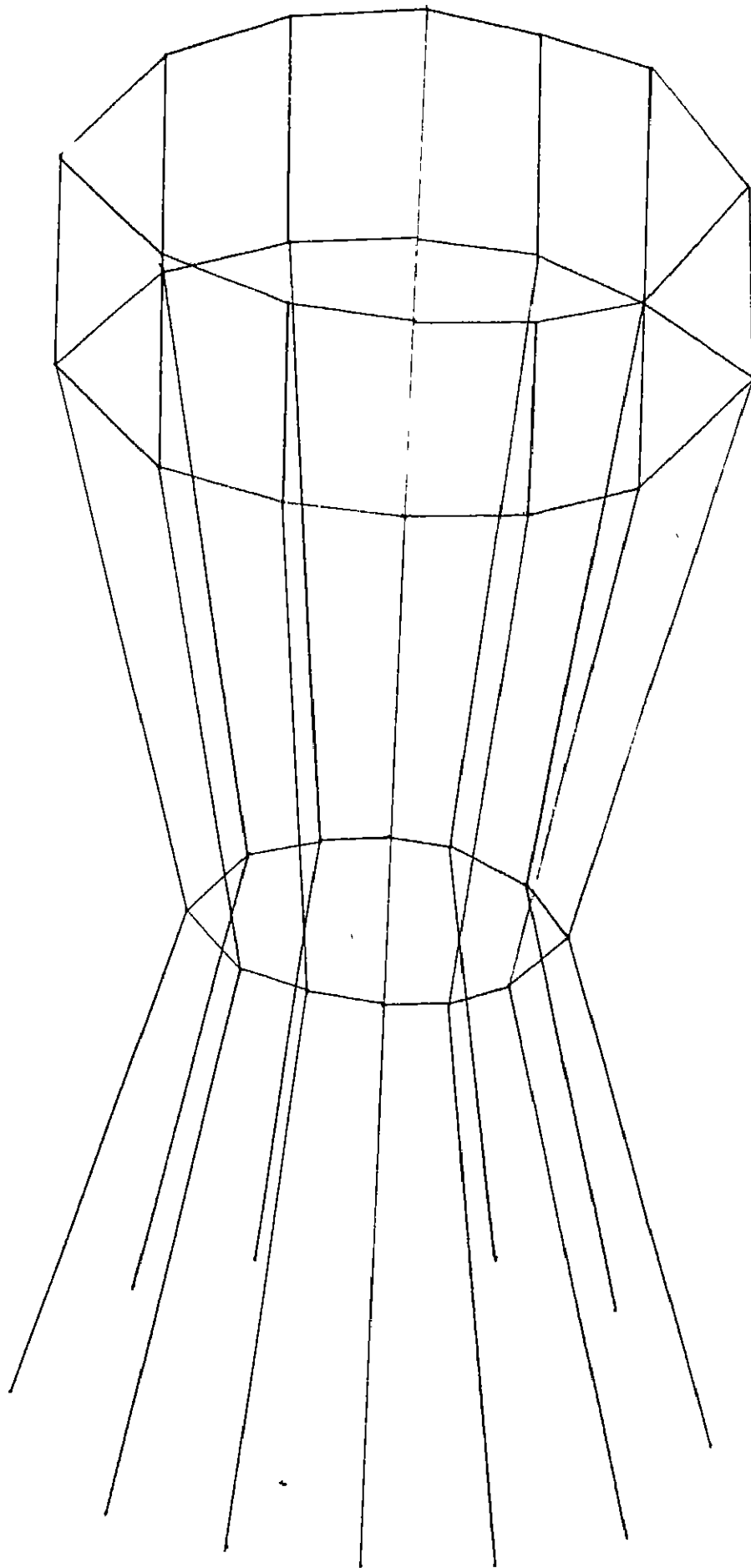


Fig-18 DIAGRAM FOR DETAILING OF REINFORCEMENT
IN WALL

►



1. RESECTION OF TANK FOR ANALYSIS

4.2 STUDY CASES

The parametric study of the water tank analysis is done varying the number of columns, the height of the domes for the lower slab carrying water and the top roof & the diameter at midlanding. The cases are given as below :

Cases according to the radius at midlanding :

- 1) Radius = 8.5 m.
- 2) Radius = 9.4 m.
- 3) Radius = 10.3 m.
- 4) Radius = 11.2 m.
- 5) Radius = 12.1 m.

Cases according to the rise of dome :

- a) $f = 0.0$ m. (Flextural member)
- b) $f = 1.0$ m.
- c) $f = 2.0$ m.
- d) $f = 3.0$ m.
- e) $f = 4.0$ m.
- f) $f = 5.0$ m.

Cases according to number of columns :

- i) No. of columns = 16
- ii) No. of columns = 14
- iii) No. of columns = 12
- iv) No. of columns = 10
- v) No. of columns = 8

4-3. DESIGN PARAMETERS IN COLUMNS

No. of columns = 16																			
---Flexure---				---Flexure---				---f=1---				---f=2---				---f=3---			
CASE	P	Mx1	Mx2	P	Mx1	Mx2	P	Mx1	Mx2	P	Mx1	Mx2	P	Mx1	Mx2	P	Mx1	Mx2	
1	204	2.9	6.1	181	0.4	1.0	182	1.7	7.1	189	2.1	8.8	196	2.8	11.5				
2	204	7.2	9.0	181	4.6	3.9	182	11.8	3.0	189	11.2	1.6	196	10.9	7.4				
3	205	5.8	13.5	182	3.1	8.6	182	11.6	14.5	189	11.9	13.4	196	11.9	11.8				
4	208	6.6	16.6	183	3.9	11.7	183	16.6	23.4	190	16.8	22.5	197	17.1	20.7				
5	211	6.9	19.2	185	4.3	14.8	185	19.4	31.0	192	20.1	30.0	200	20.5	28.7				

No. of columns = 14																
1	229	3.5	7.1	203	0.8	1.4	200	1.7	7.1	207	2.3	9.2	215	3.2	12.4	
2	229	8.0	10.4	203	5.1	4.7	200	11.2	3.1	207	11.1	1.4	215	10.5	1.5	
3	231	6.9	15.1	204	3.9	9.6	200	12.0	14.8	207	12.0	13.4	215	11.8	10.8	
4	234	7.9	18.3	205	4.8	13.0	201	16.7	23.8	209	17.0	22.6	217	17.1	20.3	
5	237	8.4	21.2	207	5.4	16.3	203	19.9	32.0	211	20.4	30.5	220	20.7	28.4	

No. of columns = 12

1	263	4.1	8.6	232	1.1	2.3	223	1.9	6.9	231	2.6	9.5	241	3.9	13.4
2	263	9.0	12.1	232	5.8	5.2	223	11.3	3.4	231	10.9	11.6	241	8.6	7.4
3	265	8.3	17.2	233	4.9	11.0	223	12.7	15.4	232	12.1	13.5	241	11.6	10.3
4	268	9.5	20.7	235	6.0	14.7	224	17.2	24.6	233	17.3	22.9	243	17.2	20.0
5	272	10.2	23.8	237	6.8	18.2	227	20.5	32.5	237	20.9	31.0	247	20.9	28.3

No. of columns = 10

1	310	5.0	10.5	273	1.6	3.3	255	1.5	5.9	265	2.6	9.0	277	4.3	13.7
2	310	10.4	14.6	273	6.7	7.3	255	11.8	4.7	265	11.3	2.0	277	9.9	2.3
3	312	10.1	20.1	273	6.2	13.0	255	13.1	17.1	266	12.8	14.7	278	11.9	10.7
4	316	11.7	23.9	275	7.7	16.9	258	18.4	26.5	268	18.3	24.3	280	17.8	20.7
5	321	12.8	27.4	278	8.7	20.8	261	21.9	34.6	272	22.1	32.6	284	21.7	29.2

No. of columns = 8

1	379	6.4	13.3	332	2.3	4.8	303	0.1	1.7	316	1.4	5.4	330	3.5	11.0
2	380	12.5	18.2	332	8.1	9.5	303	13.7	9.2	316	12.7	5.9	330	11.1	0.8
3	382	12.9	24.4	334	8.2	15.8	304	15.6	22.0	317	15.0	19.1	332	13.8	14.7
4	387	14.5	12.9	336	10.1	20.4	307	21.7	31.7	320	20.9	29.1	335	20.0	25.0
5	393	16.5	32.7	340	12.9	23.9	312	25.1	40.7	326	25.0	37.7	341	24.4	33.6

५५

DESIGN PARAMETERS IN BOTTOM CIRCULAR BEAMS

CASE	M	V	A	M	V	A	M	V	A	M	V	A	M	V	A
							No. of columns = 16								
1	5	17.6	34	9	17.6	36	46	40.0	115	56	41.7	166	69	43.7	228
2	6	17.6	44	9	17.6	48	47	40.0	137	57	41.7	189	71	43.7	253
3	6	17.6	55	8	17.6	60	48	40.0	158	59	41.7	212	73	43.7	278
4	7	17.6	64	7	17.6	71	49	40.0	181	59	41.7	237	74	43.7	304
5	8	17.6	74	7	17.6	82	49	40.0	205	60	41.7	263	74	43.7	331

No. of columns = 14															
1	9	20.6	37	7	20.6	40	61	45.7	119	72	47.7	170	88	50.0	232
2	9	20.6	49	7	20.6	53	62	45.7	140	73	47.7	193	89	50.0	257
3	10	20.6	60	6	20.6	65	63	45.7	162	75	47.7	217	91	50.0	282
4	11	20.6	70	6	20.6	78	64	45.7	185	76	47.7	241	92	50.0	308
5	11	20.6	81	5	20.6	90	64	45.7	209	76	47.7	267	93	50.0	335

No. of columns = 12

1	14	24.5	41	5	24.5	44	86	53.3	124	99	55.6	174	117	58.2	236
2	14	24.5	54	4	24.5	58	87	53.3	146	100	55.6	197	119	58.2	261
3	15	24.5	66	4	24.5	72	88	53.3	168	102	55.6	222	120	58.2	287
4	16	24.5	78	3	24.5	86	89	53.3	192	103	55.6	247	121	58.2	313
5	17	24.5	89	2	24.5	96	89	53.3	215	103	55.6	272	122	58.2	340

No. of columns = 10

1	23	30.1	47	1	30.1	49	132	54.0	132	147	66.8	182	169	70.0	243
2	24	30.1	60	2	30.1	65	133	54.0	155	149	66.8	206	171	70.0	269
3	24	30.1	74	3	30.1	80	134	54.0	177	150	66.8	230	172	70.0	296
4	26	30.1	87	3	30.1	96	134	54.0	201	151	66.8	255	173	70.0	321
5	27	30.1	100	4	30.1	111	135	54.0	225	151	66.8	281	174	70.0	349

No. of columns = 8

1	40	38.5	53.4	14	38.5	56	227	80.0	149	247	83.5	196	274	87.4	256
2	41	38.5	69	14	38.5	75	228	80.0	173	248	83.5	221	276	87.4	282
3	42	38.5	85	15	38.5	93	229	80.0	196	250	83.5	246	278	87.4	309
4	43	38.5	99	16	38.5	110	230	80.0	220	251	83.5	271	279	87.4	336
5	45	38.5	114	17	38.5	127	230	80.0	244	251	83.5	297	280	87.4	363

4.5

DESIGN PARAMETERS IN TOP CIRCULAR BEAMS

CASE	N	V	A	M	V	A	M	V	A	M	V	A			
No. of columns = 16															
1	4	6.0	33	4	6.0	33	14	17.6	45	14	19.1	65	14	20.7	88
2	4	6.0	34	4	6.0	33	14	17.6	47	14	19.1	67	20	20.7	90
3	4	6.0	36	4	6.0	35	14	17.6	49	14	19.1	69	14	20.7	92
4	4	6.0	38	4	6.0	38	14	17.6	51	14	19.1	71	14	20.7	93
5	4	6.0	41	4	6.0	42	14	17.6	52	14	19.1	72	14	20.7	94
No. of columns = 14															
1	6	6.9	36	6	6.9	35	20	20.1	41	20	21.9	62	20	23.7	86
2	6	6.9	37	6	6.9	36	20	20.1	44	20	21.9	64	20	23.7	88
3	6	6.9	38	6	6.9	38	20	20.1	46	20	21.9	66	20	23.7	89
4	6	6.9	41	6	6.9	41	20	20.1	47	20	21.9	67	20	23.7	90
5	6	6.9	44	6	6.9	45	19	20.1	49	20	21.9	68	20	23.7	91

4.5															
No. of columns = 12															
1	8	8.0	39	8	8.0	39	30	23.5	37	30	25.5	58	31	27.6	82
2	8	8.0	40	8	8.0	40	29	23.5	38	30	25.5	60	31	27.6	83
3	8	8.0	42	8	8.0	42	29	23.5	40	30	25.5	61	31	27.6	85
4	8	8.0	45	8	8.0	45	29	23.5	42	30	25.5	62	31	27.6	86
5	8	8.0	48	8	8.0	49	29	23.5	43	30	25.5	63	30	27.6	87

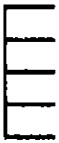

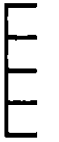
No. of columns = 10															
1	12	9.6	43	12	9.6	42	47	28.2	27	49	30.5	50	51	33.1	75
2	12	9.6	44	12	9.6	43	47	28.2	29	49	30.5	51	50	33.1	76
3	12	9.6	46	12	9.6	46	47	28.2	30	49	30.5	53	50	33.1	77
4	12	9.6	49	12	9.6	49	47	28.2	32	49	30.5	54	50	33.1	78
5	12	9.6	52	12	9.6	53	47	28.2	33	49	30.5	54	50	33.1	79

No. of columns = 8															
1	18	12.1	47	18	12.1	47	85	35.2	8	89	38.2	34	92	41.4	61
2	18	12.1	48	18	12.1	48	85	35.2	10	89	38.2	35	92	41.4	62
3	18	12.1	50	18	12.1	51	85	35.2	11	89	38.2	36	92	41.4	63
4	18	12.1	54	18	12.1	54	85	35.2	12	88	38.2	37	92	41.4	63
5	18	12.1	57	18	12.1	59	85	35.2	13	88	38.2	37	92	41.4	64

The sizes of the members are for the tank supported on less number of columns are assumed to be greater because each member will carry more load if the number of columns are less. This is assumed in case of beam and column sections only, because the thickness of the walls will primarily depend on the depth of water & thickness of bottom slab will depend primarily on the rise of dome and depth of water.

4.6

DESIGN OF TOP ROOF

Rise (f)		1.0	2.0	3.0	4.0	5.0
Radius (r)		12.0	12.0	12.0	12.0	12.0
Radius (R)		72.5	37.0	25.5	20.0	16.9
		9.53	18.92	28.07	36.87	45.24
Sin		0.17	0.32	0.47	0.60	0.71
Depths		200.0 175.0 150.0 125.0 100.0	175.0 150.0 125.0 100.0 75.0	150.0 125.0 100.0 75.0	125.0 100.0 75.0	100.0 75.0
D.L.		5.0 4.375 3.75 3.125 2.5	4.375 3.75 3.125 2.5 1.875	3.75 3.125 2.5 1.875	3.125 2.5 1.875	2.5 1.875
L.L.		0.75	0.75	0.75	0.75	0.75
TOTAL		2619 2358 2050 1765 1480	2383 2092 1802 1511 1220	2163 1863 1562 1262	1948 1634 1320	1725 1394

N		209	97	61	43	32
		189	86	53	36	26
		165	74	44	29	
		141	62	36		
		119	49			
N (min)		240	210	180	150	120
		210	180	150	120	90
		180	150	120	90	
		150	120	90		
		120	90			

So provide for all cases minimum thickness of 75 mm. may be provided. Also minimum reinforcement of 0.3% may be provided in both directions.

47

DESIGN OF BOTTOM DOME

Rise (f)		1.0	2.0	3.0	4.0	5.0
Radius (r)		12.0	12.0	12.0	12.0	12.0
Radius (R)		72.5	37.0	25.5	20.0	16.9
		9.53	18.92	28.07	36.87	45.24
Sin		0.17	0.32	0.47	0.60	0.71
Depths		400.0	375.0	350.0	325.0	300.0
		375.0	350.0	325.0	300.0	275.0
		350.0	325.0	300.0	275.0	250.0
		325.0	300.0	275.0	250.0	225.0
		300.0	275.0	250.0	225.0	200.0
D.L.		10.0	9.375	8.75	8.175	7.5
		9.375	8.75	8.175	7.5	6.875
		8.75	8.175	7.5	6.875	6.25
		8.175	7.5	6.875	6.25	5.625
		7.5	6.875	6.25	5.625	5.0
L.L.		20000	20000	20000	20000	20000
TOTAL		24555	24359	24205	24109	23982
		24271	24068	23929	23770	23650
		23986	23801	23605	23456	23318
		23701	23487	23304	23142	22986
		23416	23173	23004	22827	22655

N	[1960	996	682	533	448
		1945	984	674	525	441
		1922	973	665	518	436
		1899	960	657	512	429
		1876	948	648	504	423
Ast	[25.5	10.0	5.2	3.1	2.0
		27.7	11.2	6.1	3.8	2.8
		29.8	12.5	7.1	4.5	3.8
		32.2	13.9	8.3	5.9	4.9
		34.1	15.6	9.7	7.2	6.4

4.8

DESIGN OF TOP RING BEAM

	1	2	3	4	5
Rise (f)	1.0	2.0	3.0	4.0	5.0
Radius (r)	12.0	12.0	12.0	12.0	12.0
	9.53	18.92	28.07	36.87	45.24
Tension	1188	593	394	293	230
Ast	7920	3953	2627	1953	1533
Steel	10-32	6-25	6-25	4-20	2-20
		+		+	+
Beam sizes	750	700	600	500	400
	X	X	X	X	X
	750	700	600	500	400
pt (%)	1.429	0.857	0.818	0.820	0.895
Beam vol.	43.30	37.71	27.71	19.24	12.31

4.9

DESIGN OF BOTTOM RING BEAM

	1	2	3	4	5
Rise (f)	1.0	2.0	3.0	4.0	5.0
Radius (r)	12.0	12.0	12.0	12.0	12.0
	9.53	18.92	28.07	36.87	45.24
Tension	23195	11306	7721	5116	3785
Ast	154633	75373	51473	34106	25236
Steel (32)	194	92	64	44	34
pt (%)	10.4	5.04	3.43	2.36	1.80
Beam vol.	117.81	117.81	117.81	117.81	117.81

4.10 DESIGN OF FOUNDATION

Rise (mid)	Dia.	COL.-	8	10	12	14	16
4	12.1	W:-	346	289	251	225	204
4	11.2	W:-	346	289	251	225	204
4	10.3	W:-	348	290	252	225	204
4	9.4	W:-	352	293	254	226	205
4	8.5	W:-	358	298	256	229	208
5	12.1	W:-	363	302	263	234	213
5	11.2	W:-	363	302	263	234	213
5	10.3	W:-	366	304	264	234	213
5	9.4	W:-	370	307	266	236	214
5	8.5	W:-	376	313	270	240	218

W : Total load including self weight.

No. of piles	7	6	5	5	4
Sizes (m x m)	5.8X3.3	5.8X3.3	3.3X3.3	3.3X3.3	3.3X3.3
Column(m x m)	1.2X0.65	1.1X0.6	1.0X0.55	0.9X0.5	0.8X0.45
Pile vol.	112	120	120	140	128
Pile cap vol.	132	159	112	112	140
Total vol.	244	279	232	262	268

4.11 DESIGN OF COLUMNS

For the columns the following sizes were taken. It gave reinforcement within specified limit / 22 /. The volume of the columns for a height of 20 m. are also being calculated.

No.	8	10	12	14	16
Sizes(mxmxm)	1.2X0.65	1.1X0.6	1.0X0.55	0.9X0.5	0.8X0.45
Area	0.78	0.66	0.55	0.45	0.36
Total area	6.24	6.60	6.60	6.30	5.76
Volume	124.8	132	132	126	115.2

4.12 DESIGN OF TANK WALLS

Calculation of level water in the tank

Rise (f)	1.0	2.0	3.0	4.0	5.0
Vol 1	226	457	693	938	1196
Vol 2	226	448	664	871	1065
Vol 3	1774	1552	1336	1129	935
Level 1	3.92	3.43	2.95	2.50	2.07
Level 2	4.92	5.43	5.95	6.50	7.07
Level 3	5.3	5.8	6.3	6.8	7.4

VOL 1 : Volume below the dome curve
 VOL 2 : Volume above the dome curve
 VOL 3 : Extra volume of water
 Level 1 : Water level for extra water
 Level 2 : Total depth of water
 Level 3 : Total depth of wall

4.12.1 CALCULATION OF MINIMUM THICKNESS OF WALL AT BASE

RISE (f)	1.0	2.0	3.0	4.0	5.0
W.L. (a)	5.0	5.5	6.0	6.5	7.1

Number of columns : 16 --> b = 4.26 m.

b/a	0.852	0.775	0.710	0.655	0.600
α_n	0.0285	0.0251	0.0226	0.0206	0.0186
Moment	35.625	41.76	48.82	56.57	66.57
D max.	355	384	415	447	485
D req.	350	375	400	425	475

Number of columns : 14 --> b = 4.88 m.

b/a	0.976	0.887	0.813	0.751	0.687
α_n	0.0339	0.030	0.0268	0.0241	0.0217
Moment	42.375	49.91	57.89	66.18	77.67
D max.	387	420	452	483	524
D req.	375	400	425	475	500

Number of columns : 12 --> b = 5.73 m.

b/a	1.146	1.042	0.955	0.882	0.807
α_n	0.042	0.037	0.033	0.0298	0.0265
Moment	52.5	61.56	71.28	82.84	96.85
D max.	430	466	502	541	584
D req.	425	450	475	525	550

Number of columns : 10 --> b = 6.84 m.

b/a	1.368	1.244	1.140	1.052	0.963
α_n	0.053	0.0467	0.0417	0.0375	0.0334
Moment	66.25	77.70	90.07	102.98	119.54
D max.	484	524	564	603	650
D req.	475	500	550	575	625

Number of columns : 8 --> b = 8.67 m.

b/a	1.737	1.576	1.445	1.334	1.220
α_n	0.0733	0.0643	0.0571	0.0514	0.0456
Moment	91.625	106.98	123.42	141.16	163.21
D max.	569	614	660	706	759
D req.	550	575	625	675	725

TABLE - 4

TABLE -5

4.12.2 CALCULATION OF MINIMUM THICKNESS OF WALL AT MIDDLE LEVEL

RISE (f)	1.0	2.0	3.0	4.0	5.0
W.L. (a)	5.0	5.5	6.0	6.5	7.1

Number of columns : 16 --> b = 4.16 m.

b/a	0.852	0.775	0.710	0.655	0.600
α_n	0.0217	0.0182	0.0157	0.014	0.0122
Moment	27.375	30.28	33.91	38.45	43.67
D max.	310	327	346	368	393
D req.	300	325	325	350	375

Number of columns : 14 --> b = 4.88 m.

b/a	0.976	0.887	0.813	0.751	0.687
α_n	0.0278	0.0235	0.0200	0.0171	0.0150
Moment	34.75	39.10	43.20	46.96	53.69
D max.	350	371	390	407	435
D req.	325	350	375	400	425

Number of columns : 12 --> b = 5.73 m.

b/a	1.146	1.042	0.955	0.882	0.807
α_n	0.0337	0.0303	0.0268	0.0233	0.0197
Moment	42.125	50.41	57.89	63.99	70.51
D max.	386	422	452	475	499
D req.	375	400	425	450	475

Number of columns : 10 --> b = 6.84 m.

b/a	1.368	1.244	1.140	1.052	0.963
α_n	0.0394	0.0368	0.0335	0.0307	0.0272
Moment	49.25	61.23	72.36	84.31	97.35
D max.	417	465	505	545	586
D req.	400	450	475	525	550

Number of columns : 8 --> b = 8.67 m.

b/a	1.737	1.576	1.445	1.334	1.220
α_n	0.0516	0.0464	0.0418	0.0387	0.0360
Moment	64.5	77.29	90.29	106.28	128.85
D max.	477	522	564	612	674
D req.	450	500	550	600	650

Here the moments are in kNm/m and the depths are in mm. The reinforcements are varying from 0.2% (min.) to 0.5%. Calculation of depth and reinforcements taking only

tension

TABLE-6

4.12.3 DESIGN OF TANK WALL AS A TENSION MEMBER

RISE (f)	1.0	2.0	3.0	4.0	5.0
W.L. (a)	5.0	5.5	6.0	6.5	7.1
Wn (kN/m/m)	50.0	55.0	60.0	65.0	71.0
T (kN/m)	600.0	660.0	720.0	780.0	852.0
Ast	4000.0	4440.0	4800.0	5200.0	5680.0
Depth	500.0	550.0	600.0	650.0	710.0
p (%)	0.63	0.63	0.63	0.63	0.63

So the tank walls should be designed as two way slab i.e. as flextural member to reduce the depth as well as percentage of steel.

TABLE-7

4.12.4 VOLUME OF TANK WALL FOR DIFFERENT CASES

RISE (f)	1.0	2.0	3.0	4.0	5.0
HEIGHT	5.3	5.8	6.3	6.8	7.4
16 Column	132.903	156.630	178.639	203.310	242.659
14 Column	143.129	167.817	194.437	229.544	264.091
12 Column	163.574	190.193	218.741	255.777	292.620
10 Column	178.155	212.569	249.122	288.569	335.442
8 Column	204.562	240.538	285.579	334.477	392.539

4.13 CALCULATION

4.13.3 Calculation for designed tank

Amount of reinforcement used

Dia. (mm)	Length (m)	Vol. cu m.
6	3307.71	0.0093
8	30033.63	1.5092
10	28171.40	2.2126
12	45193.43	5.1113
16	7831.39	1.5745
20	6072.44	1.9077
25	6668.65	3.2735
32	6451.07	5.1883

TOTAL VOLUME --> 20.8712 m³

Volume of concrete to be used in the tank

Substructure :

Below column - 195
Below shaft - 109
Tension beam - 12.3
Piles - 60.5
TOTAL - 376.8

Superstructure :

Weight/unit wt. = 20124.18/25
= 804.26

TOTAL :: 1181.76 m³

4.13.2 Calculation for optimum shape

A rise of less than 4 m. in the bottom dome increases the steel percentage beyond the permissible limit specified / 22 /. So this cases are neglected. Again if the midlanding diameter is decreased circular beam or radial beams have to be provided which will increase the cost. So straight columns are considered for staging.

Calculation of volume :

No. of columns :	8	10	12	14	16
Vol. of column	124.8	132	132	126	115.2
Vol. of wall*	334	288	255	229	203
Vol. of wall**	392	335	292	264	242
Vol. of foundation	244	279	232	262	268
Vol. of beams*	136	136	136	136	136
Vol. of beams**	130	130	130	130	130
Vol. of domes*	187	187	187	187	187
Vol. of domes**	167	167	167	167	167
Total *	1026	1023	943	941	920
Total **	1067	1053	926	959	942

* - For a rise of 4 m.

** - For a rise of 5 m.

Therefore saving that could have been done is
 3
 around 261 m³ i.e., 28.6 %.

4.14 STUDY OF RESULTS

4.14.1 DESIGN OF ROOF SLAB

Design parameters of shell structure generally depends on the curvature of the surface and the loadings. In case circular domes the design depends on the radius at the base and the height of rise of the dome. In this case the design depends primarily on the rise of the dome as the radius is fixed. Here due to very small amount of load a minimum depth 75 mm and a minimum reinforcement of 0.30% is sufficient enough for the serviceability of the system.

In comparison, in the flat flextural slab the depth depends on the length of the beams on which it is supported which depends on the number of columns on which the beams are supported. For the present problem a minimum depth of 150 mm is required along with an intermediate circular beam.

4.14.2 DESIGN OF BOTTOM SLAB

It has been shown earlier that design of bottom slab is very related with the rise of dome. For a radius of 12 m the suitable rise is 4 m & 5 m. For less rise there is a steep increase in the requirement of steel. The percentage of steel goes up from a moderate value of 2.04% at a rise of 5.0 m to an unacceptable value of 35.1% at a rise of 1.0 m while the depth is kept at a fixed value of 300 mm. Even if the depth is increased to 400 mm the requirement of steel

comes down only to 28%. So it is always adviseable to have more rise in domes. The recommended value is of the order of one-sixth to one-seventh of the base diameter (1).

4.14.3 DESIGN OF TANK WALL

The depth of tank wall varies with the rise of the dome and the radius of curvature of the surface.

It has been shown that the material requirement is less for the design of the wall as flexural member supported on the beam and columns. So naturally the depth of depends on the beams while other parameters are fixed. And the length of beams depends on the number of columns on which it is supported.

Here it is adviseable to use less rise of the dome, because the increase in the rise of dome increases the depth of water in the tank. This increases the loaded area in the slab which in turn increases the depth of the slab. For the present case it would be better to take rise of 4 m.

4.14.4 DESIGN OF TOP CIRCULAR BEAM

Analysis by F.E.M. is less accurate here because the curved members are taken as straight members and that too is defined by two points only. Here the circular members

are designed as ring beam under tension. Due to very less load the rise of the dome has a very little effect in the design of the beams.

4.14.5 DESIGN OF BOTTOM CIRCULAR BEAM

The bottom circular beam is also designed as ring beam under tension. Here the percentage of steel requirements are very high in case of low rise of the domes. Axial forces in the beams are considerably less in case of F.E.M. analysis because of idealization of the members.

Here also it is advisable to use higher rise in the dome to reduce the percentage of steel keeping the beam size fixed.

4.14.6 DESIGN OF COLUMNS

It is seen here that the axial force in the columns depends on the diameter at midlanding, rise of dome, number of columns in the systems. It is recommended here (based on the results) to have vertical columns because the design parameters increases in inclined columns. The gradient is small at the beginning and increases rapidly afterwards (fig.-). It is also recommended to have two more bracings in two more levels because relatively higher additional moment has to be considered in minor axis where the moments in the major axis is nearer to the moment due to

Explan

Explan

minimum eccentricity.

like - 2

The trend in the curves show that if the diameter in midlanding is increased more than the diameter at base of tank the force will increase because the tension will be generated at the beams.

4.1.7 DESIGN OF FOUNDATION

It is advisable to use vertical columns for support because otherwise shear force is generated. The shear force increases with the increase of the angle of inclination of the columns. *Expt*

Design has shown that vertical columns requires least amount of material.

For the present design the increase in cost *is* primarily due to the shaft. The construction of shaft and its foundation will require large amount of material. Indirectly it also affects the design of the ~~design~~ of radial beams, because the connection of the radial beams at the shaft can't be established properly. It is neither a fixed joint nor a free joint. If joint considered fixed the value of the design parameters in the radial beams at the shaft increases very much whereas if the joint is considered as free then the value of the design parameters increases in all other members.

The presence of the radial beams increases the dead load

in the system and as a result requirement of steel increases in the columns. Again from the point of view analysis in computers the flat slab system requires more time in computers.

The only question which may be raised here regarding the safety of the system is the joint condition consideration of the radial beams at the shaft. The superstructure is designed with the higher values of the design parameters except the radial beams. The substructure and the radial beams have designed taking intermediate values.

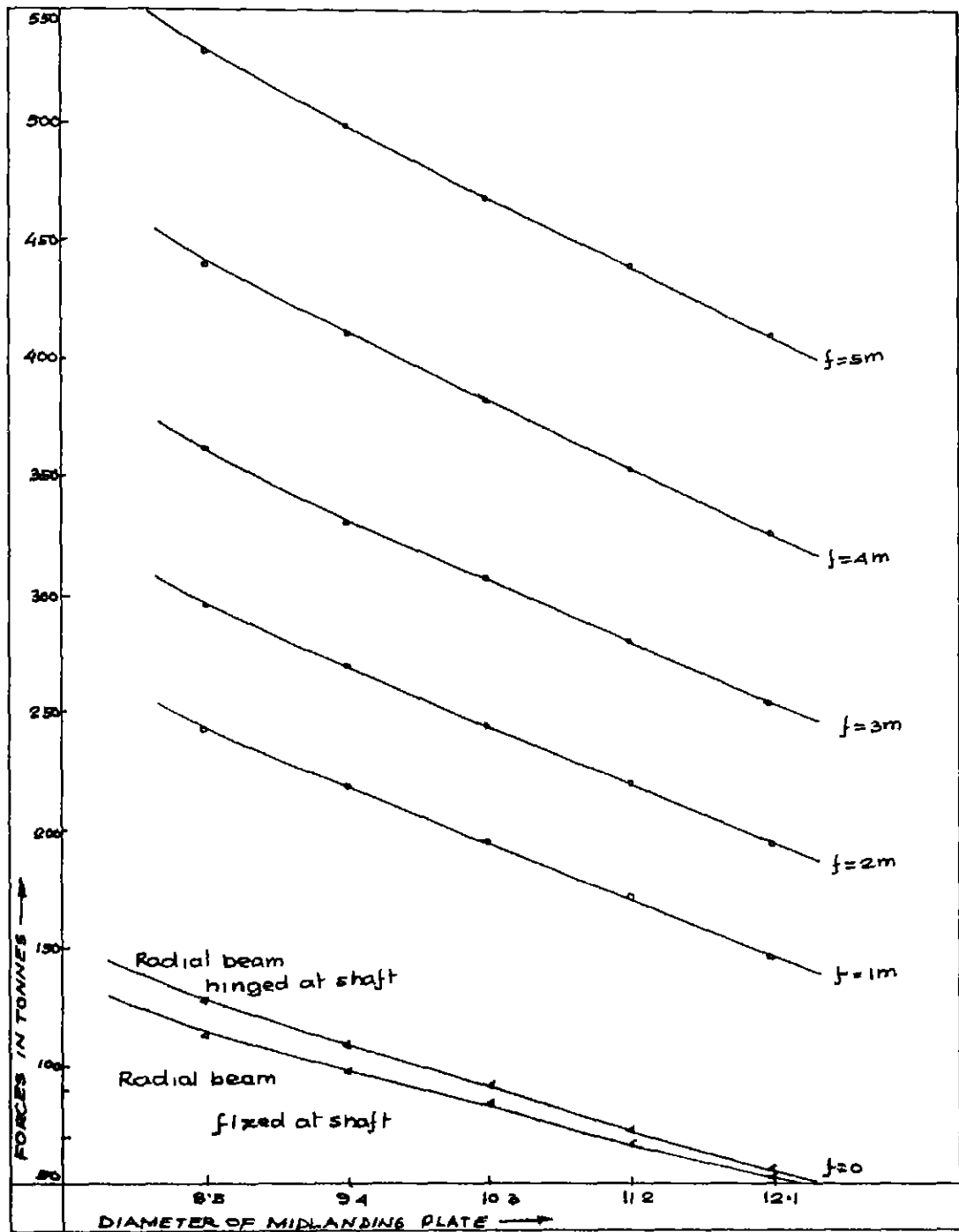


Fig - 19A Tensile force in bottom circular beam
(No. of columns = 8)

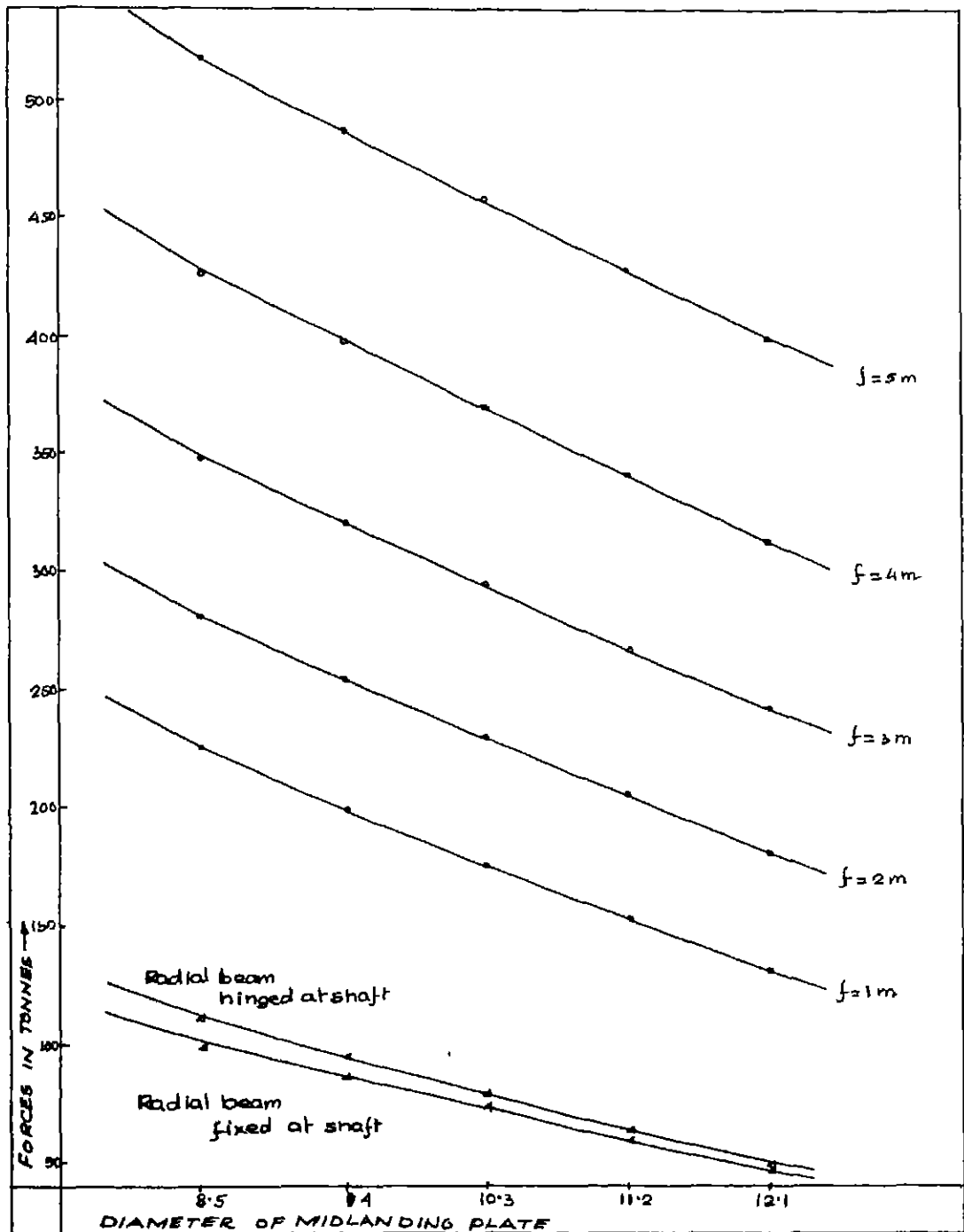


Fig.-19B TENSILE FORCE IN BOTTOM CIRCULAR BEAM
(NO of columns - 10)

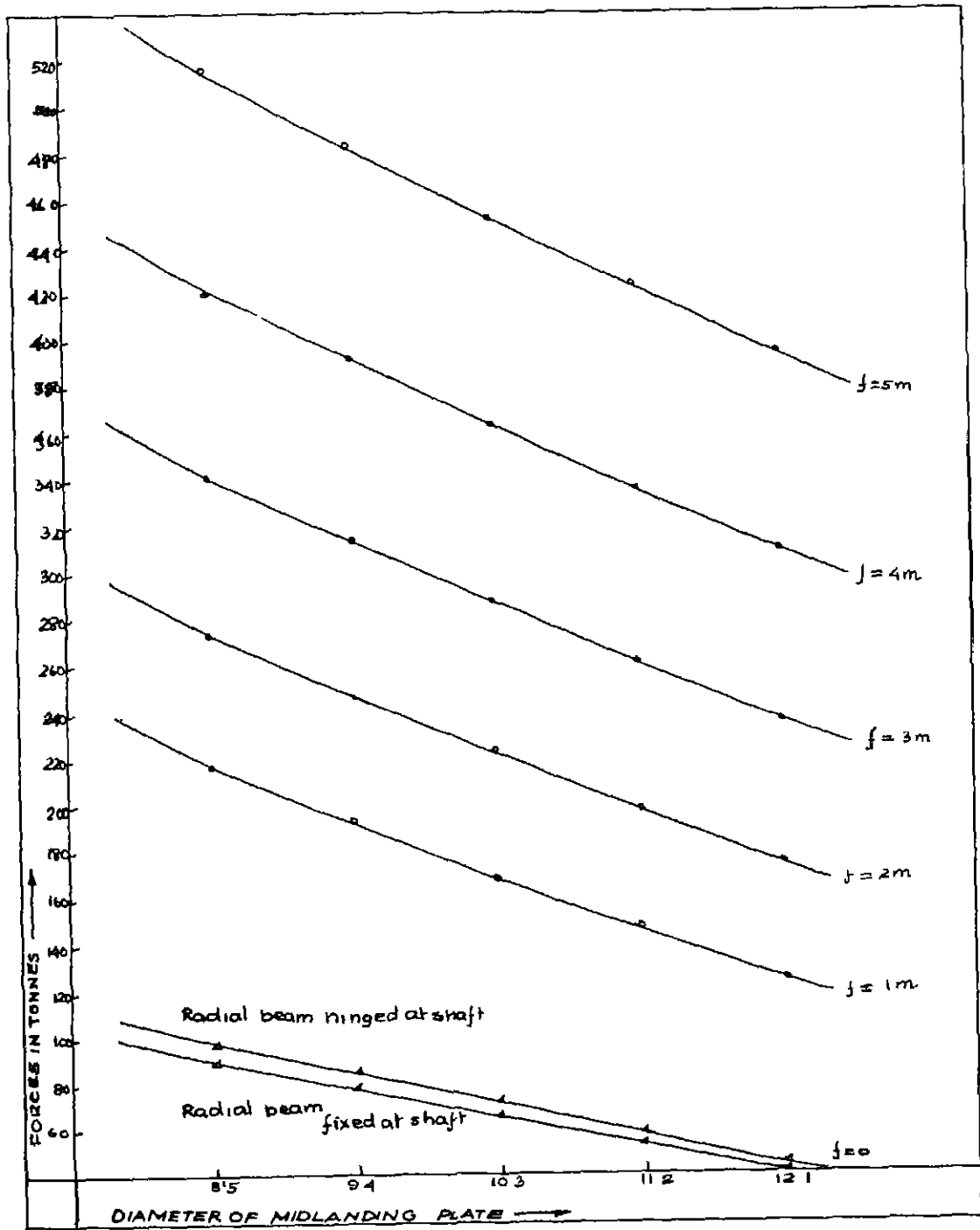


Fig - 19C Tensile force in bottom circular beam
(No of columns - 12)

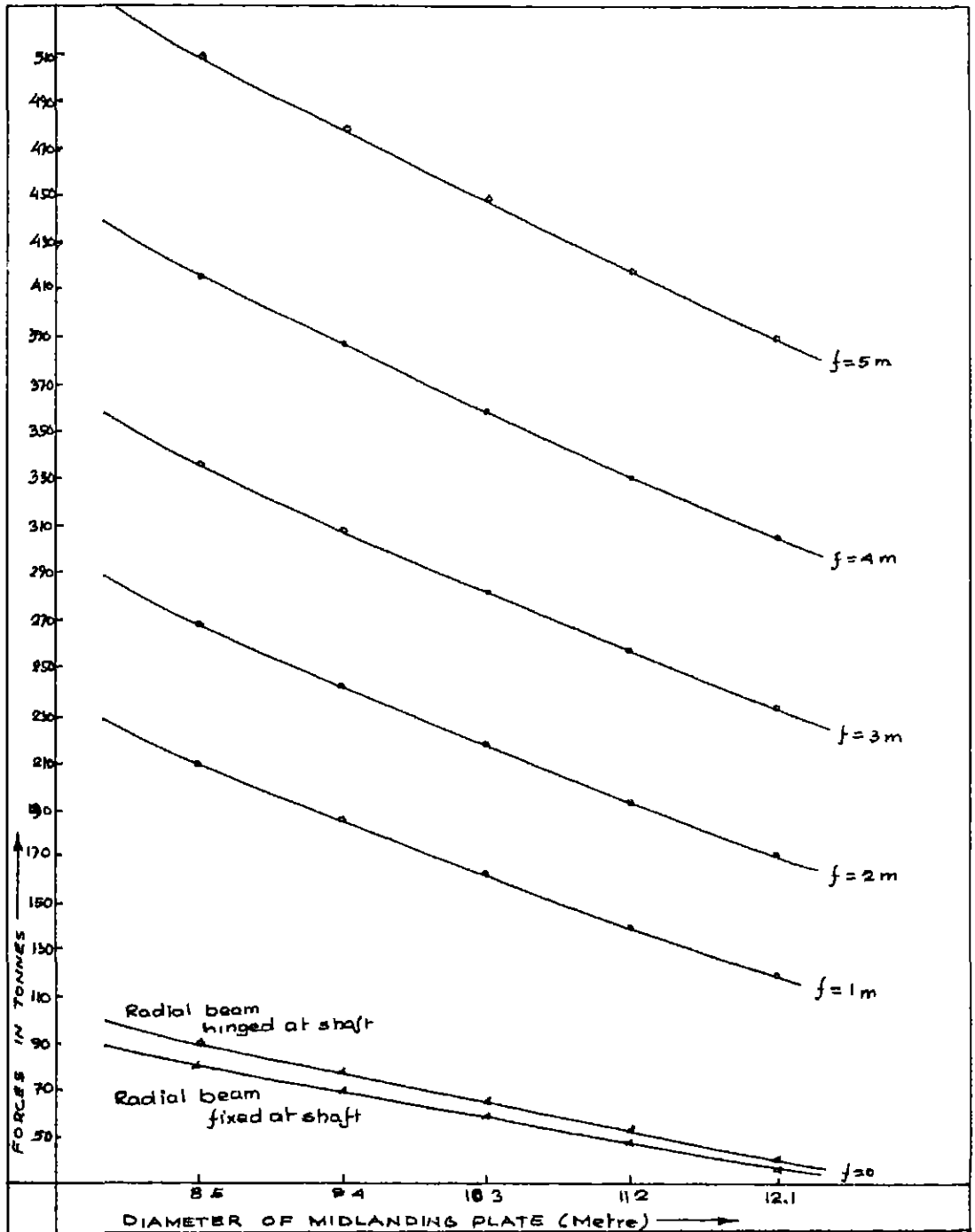


Fig.-19D : TENSILE FORCES IN BOTTOM CIRCULAR BEAM
(NUMBER OF COLUMNS - 14)

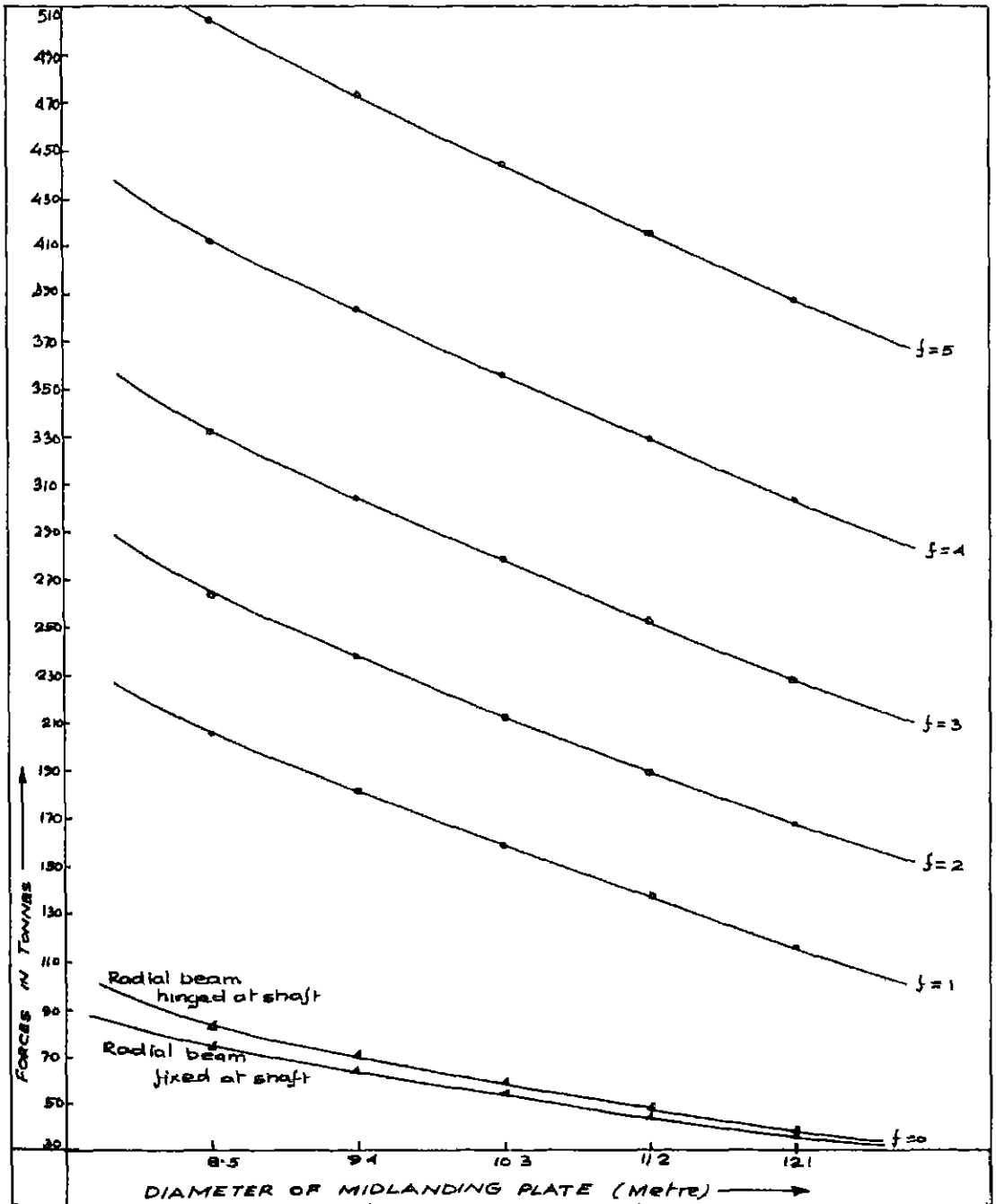


Fig - 19E . TENSILE FORCE IN BOTTOM CIRCULAR BEAM
(NUMBER OF COLUMNS = 16)

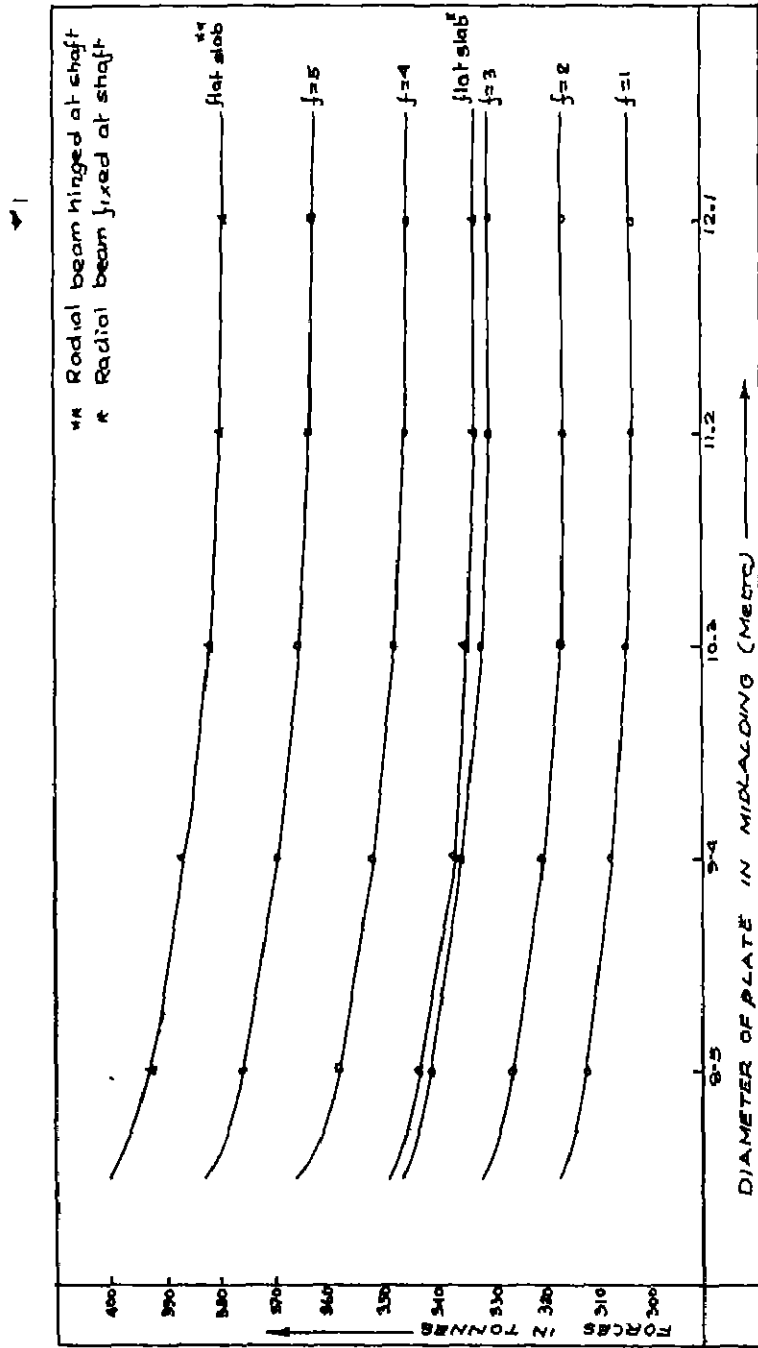


Fig.-20A ' AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOME
(NUMBER OF COLUMNS - 8)

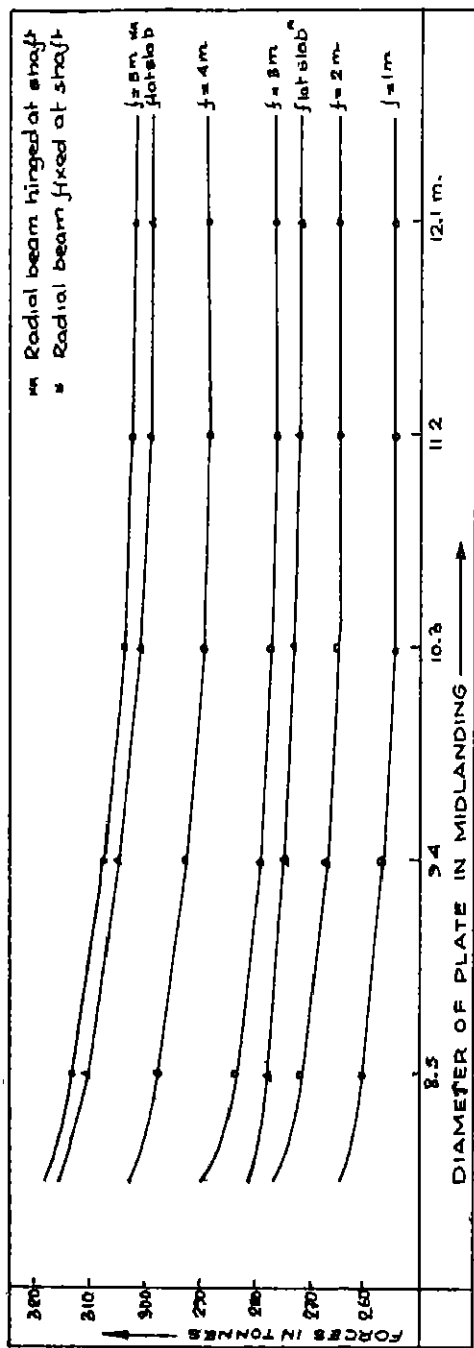


Fig. 205 : AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS-10)

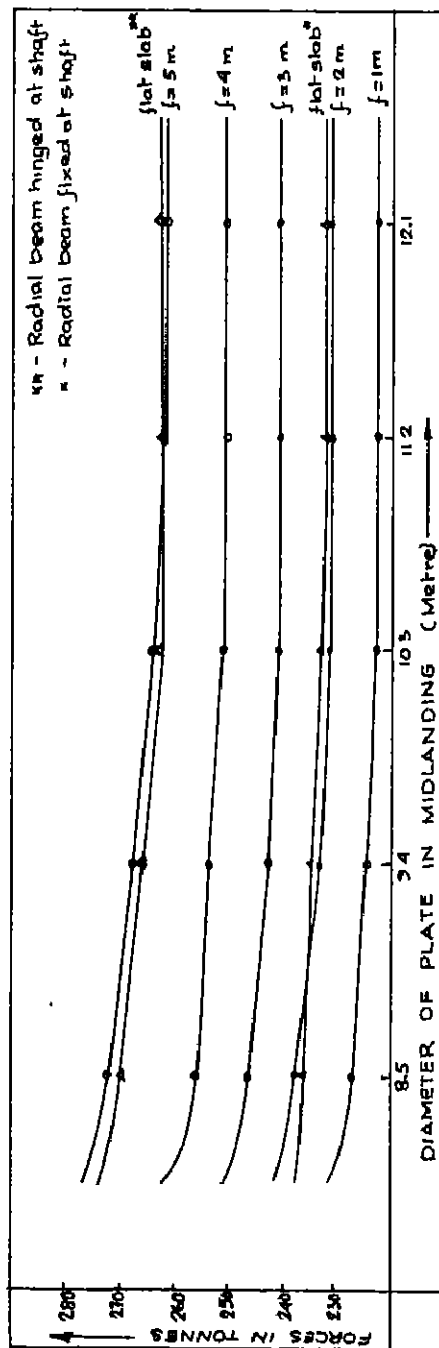


Fig. 206 : AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS-12)

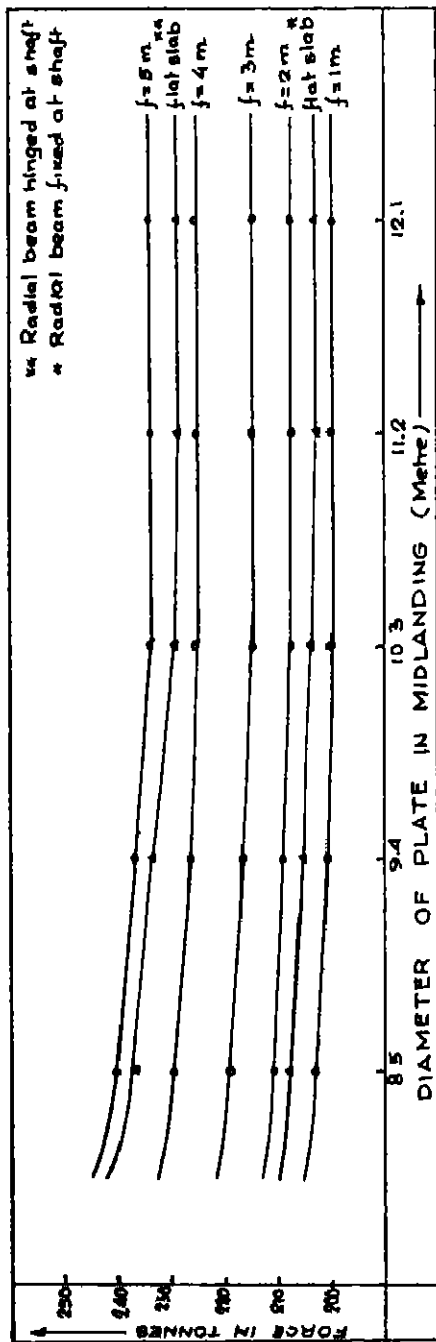


Fig-14 : AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS-14)

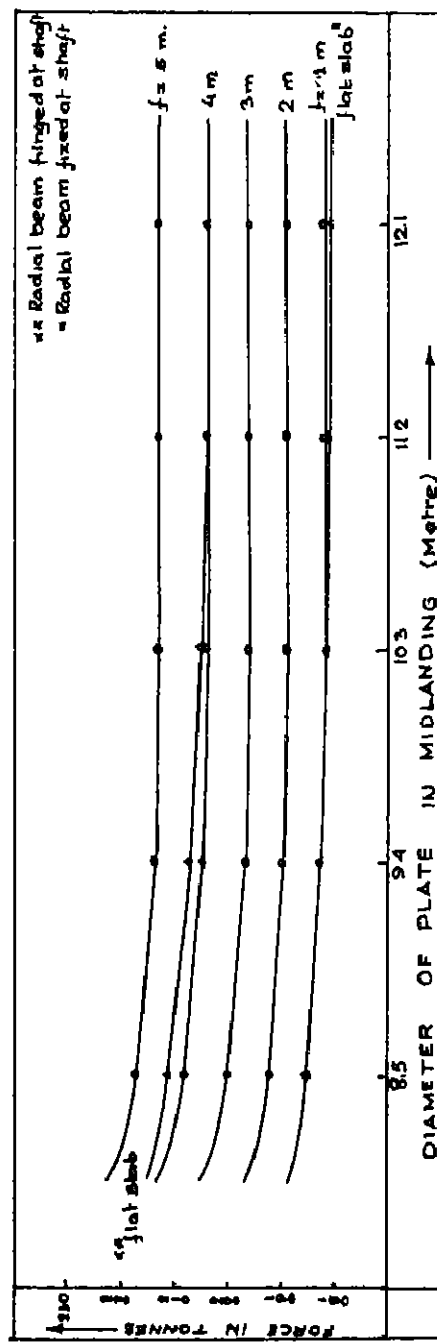


Fig-16: AXIAL FORCES IN COLUMNS WITH DIFFERENT RISES IN DOMES
(NUMBER OF COLUMNS 16)

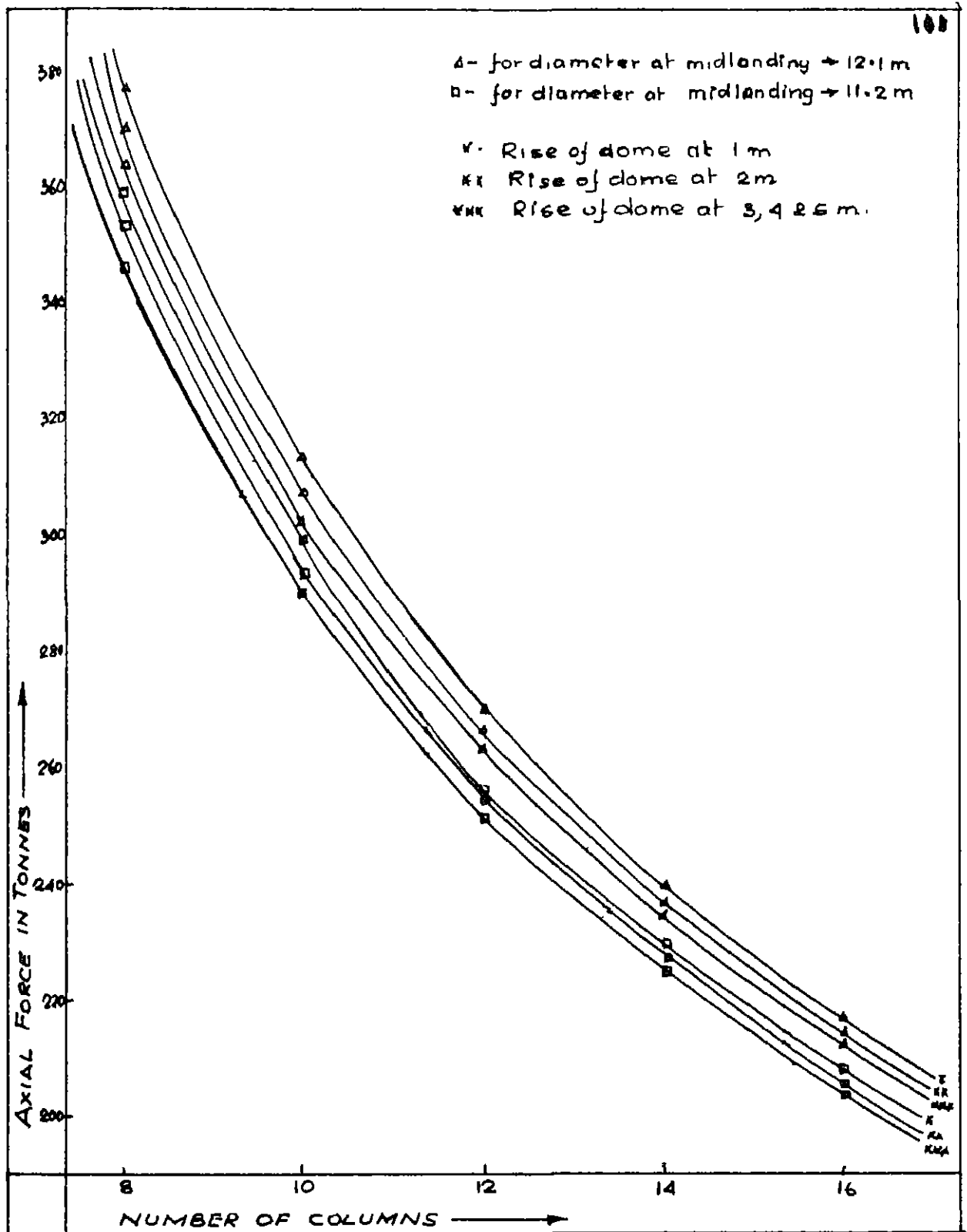


Fig- 20D : AXIAL FORCES IN COLUMNS

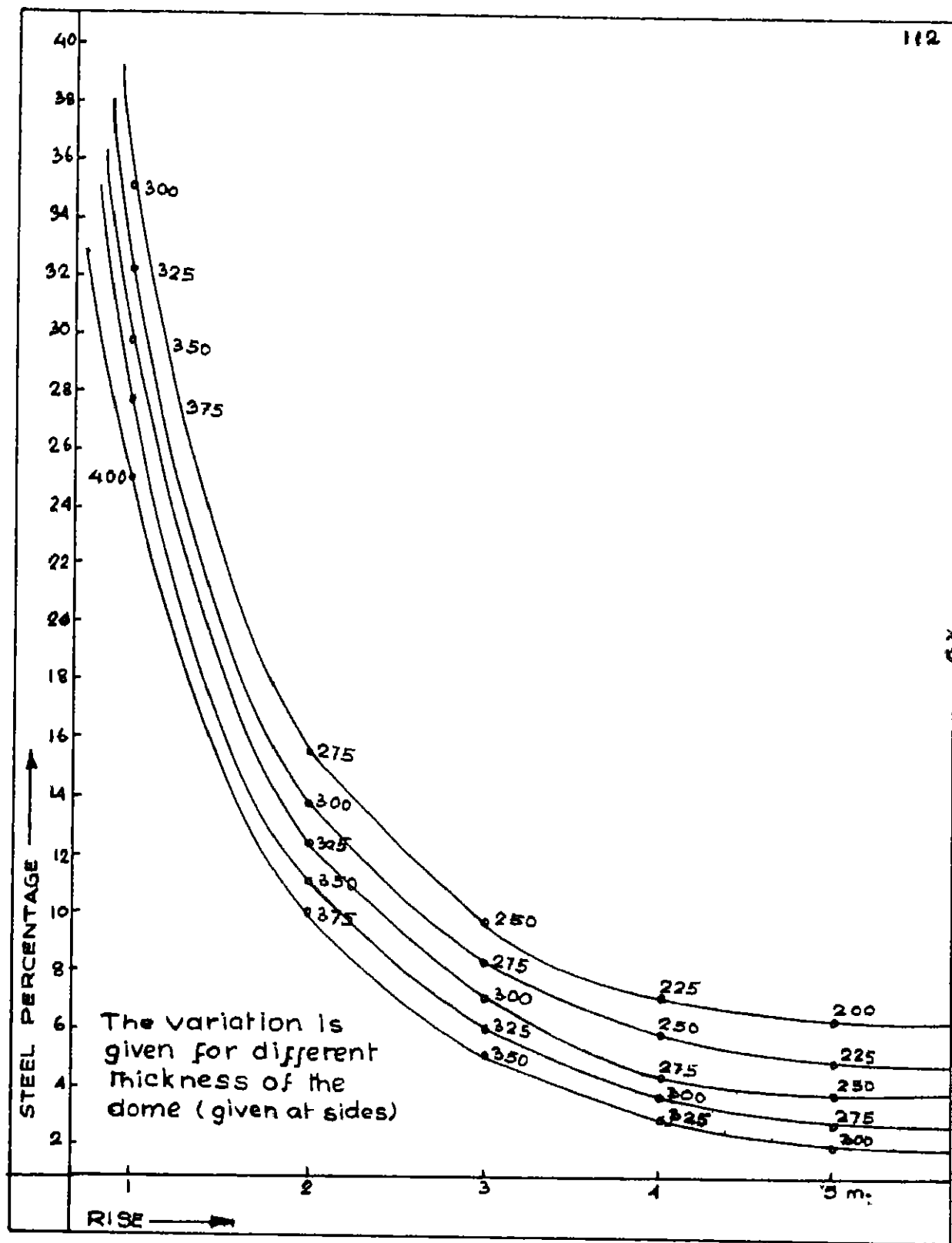


Fig.- 21 : PERCENTAGE OF STEEL IN BOTTOM DOME W.R.T. RISE OF DOME

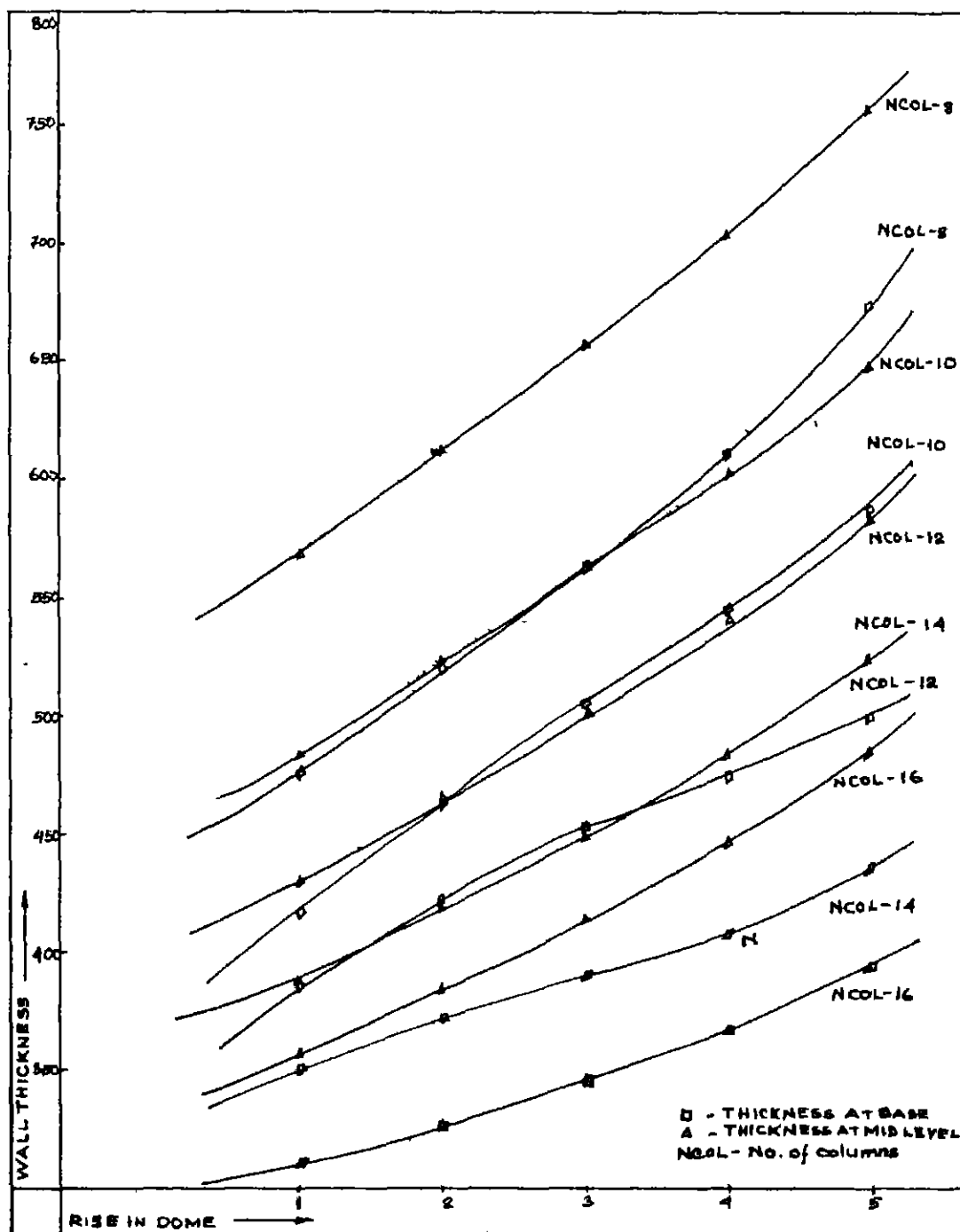


Fig - 22 - VARIATION OF TANK WALL THICKNESS
W R.T. RISE OF DOME

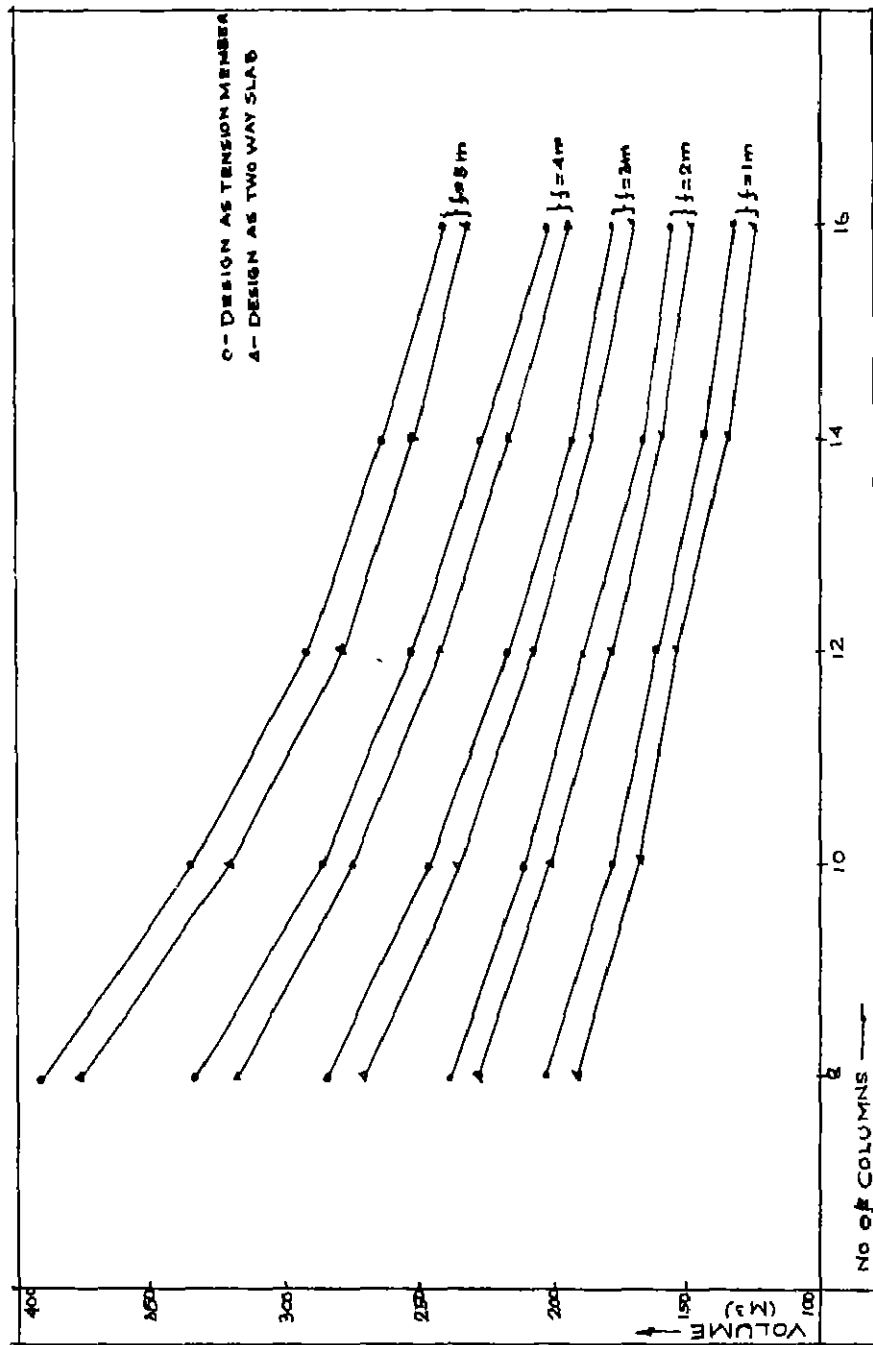


Fig -23 VOLUME OF TANK WALL FOR DIFFERENT RISES IN BOTTOM DOME

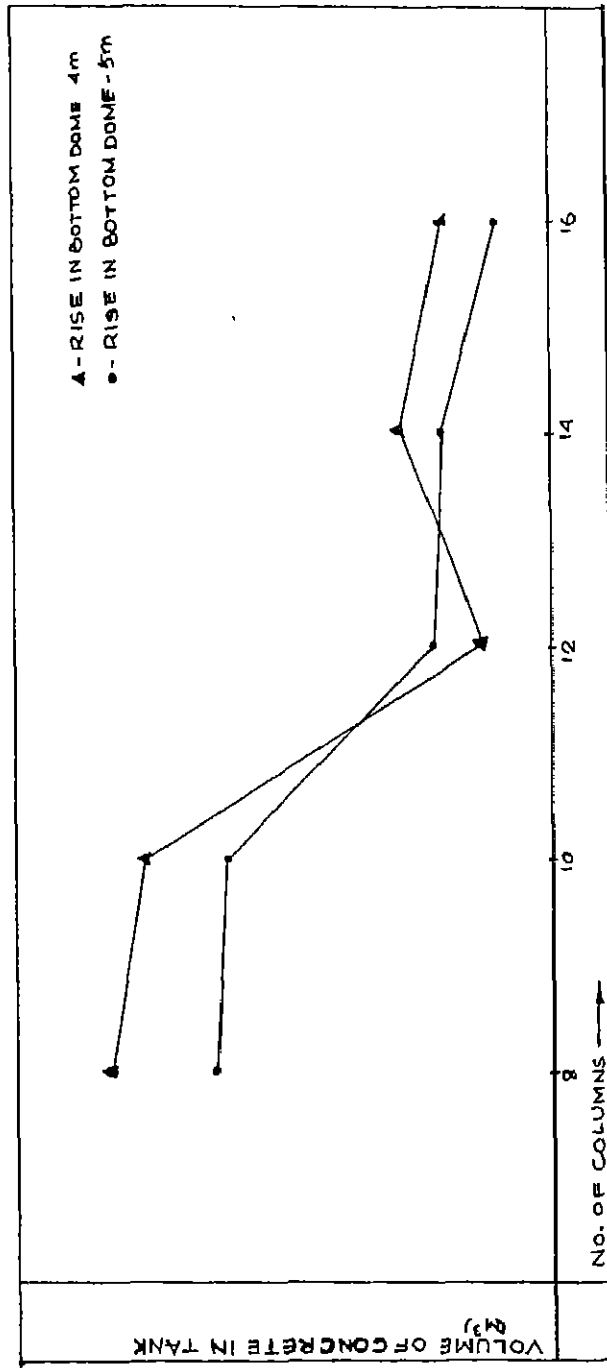


Fig-24 TOTAL VOLUME OF CONCRETE FOR DIFFERENT STAGING

OPTIMIZATION OF SHAPE OF WATER TANKS

5.1 INTRODUCTION

In designing a structure, the engineer has a choice of two basic strategies.

The traditional design procedure consists of first adopting intuitively the geometry and materials of the structures and then calculating for specified design loads the value of behavioral or state variables such as stresses, strains and displacements. After successive modification of the geometry, the procedure is then repeated until the calculated behavior satisfies certain prescribed requirements which are usually expressed in the form of inequalities representing upper limits on stresses and displacements or a lower limit of load capacity. The disadvantages of the foregoing procedure are obvious. First much computational effort may be wasted on successive analysis. Second, the design can be highly uneconomical, even when an intuitively selected solution satisfied the behavioral constraints.

To eliminate these drawbacks, the traditional procedure may be somewhat reversed. In what is termed optimal design first the required structural behavior together with design loads and geometrical constraints are specified and a further quantity termed cost or objective function is also defined. The aim of the subsequent computational effort is to select the geometry and possibly materials of the

structure, so that the required behavior is achieved at smallest possible cost.

5.2 PREVIOUS STUDIES

A shape optimization process begins by the definition of the geometry, its parameter & limitation and their dependencies on the design variables. The design of water tower shape depends on the design of cylindrical shells, conical shells, spherical domes and circular beams. To have an optimized shape of water towers all the components should be designed in such a way so that it satisfies the objective function. Here the is defined by the line of the different shells viz., cylindrical, conical, spherical joined at the points where the ring beams are holding them together.

Marcelin & Trompette / 8 / studied about optimal shape of thin axisymmetric shells. Their main aim was to obtain shapes giving rise to uniform maximum reference stress σ along the variable boundaries. The unconstrained associated minimization is solved by the variable method of Davidon-Fletcher-Powell with the help of a finite element model. And they tested the results on various examples viz., shape of a drop of water, shape of a bottle, etc.

Kapoor & Hariharan / 4 / studied about the optimal design of reinforced concrete chimneys with reference to a practical design and showed the amount of savings that could be made.

Thambiratnam et al / 14 / recommended about minimum weight design of cylindrical wall tanks. They recommended height to diameter ratio of cylindrical water tanks for smaller capacities.

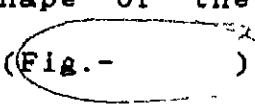
Melchers & Rozvany / 9 / considered about the optimum lower bound design of reinforced tanks. They considered circular cylindrical shell subjected to outward radial pressure without any axial (longitudinal) loading for two cases e.g., free at top & fixed at base and free at top & hinged at base. They showed that designs based on the lower bound theory the regions and region boundaries gave total lower steel volumes than equivalent design based on elastic stress distribution, or an uniform isotropic yield line analysis.

Thambiratnam et al / 15 / studied about the maximization of maximization of natural frequencies of cylindrical shells. They recommended for designing of a structure to isolate from destructive effects of resonant vibration rather than strengthening it to withstand the vibration. This is achieved by elevating the natural frequencies of the structure so that they are sufficiently higher than the frequencies of all possible dynamic excitations in the vicinity. They showed that the lowest frequency ω_0 for axisymmetric vibration kept on increasing with the number of slopes in the tank wall and limiting case was a shell whose outer wall was curve in the form of a circular arc with zero thickness at the top.

Thompson & Hunt / 17 / made a laboratory study about effect of asymmetry of an element if the element is being designed as symmetrical member. They observed that the load bearing capacity of elastic arches, cylindrical shells, complete spherical sections, complete cylindrical sections reduces with the degree of asymmetry and the that follow a curve of a hyperbola.

The problem of optimum design is defined as the determination of a design specified within a set of parameters, which minimizes certain structural property. The admissible designs from which the optimal has to chosen is called the class of safe designs.

5.3 FORMULATION OF MODEL

The primary shape of the model is taken to be a circular intze tank (Fig.- ). Having the primary shape in mind here six other cases are considered by giving some preassigned values to some of the variables.

The variables are listed as below:

1) Set A

- | | |
|-----------------------------------|------|
| a) Height of the first section | : H2 |
| b) Height of the second section | : H3 |
| c) Radius of the top ring beam | : R1 |
| d) Radius of the middle ring beam | : R2 |
| e) Radius of the bottom ring beam | : R3 |

2) Set B

- a) Radius of the shaft : R4
- b) Outer radius of the plate over shaft : R5
- c) Thickness of the plate : T6

3) Set C

- a) Height of top dome : H1
- b) Height of bottom dome : H4

4) Set D

- a) Thickness of the top dome : T1
- b) Thickness of the top conical section : T2
- c) Thickness of the bottom conical section : T3
- d) Thickness of the bottom dome : T4
- e) Thickness of shaft

The objective function here is to obtain a shape of the tank for a particular capacity so that it transfer least weight to its staging.

Five basic design variables (Set : A) have been considered here. Out of these variables height of the first conical shell is calculated from the volume relationship using the other four variables.

Values of three variables (Set : B) have been kept fixed depending on the type of tank.

Rise of the spherical domes have always been kept fixed at one sixth of the radius of the dome at base.

Thickness of the sections have been varied within the prescribed maximum and minimum values. The reinforcement is also restricted by a maximum (0.035) and a minimum value

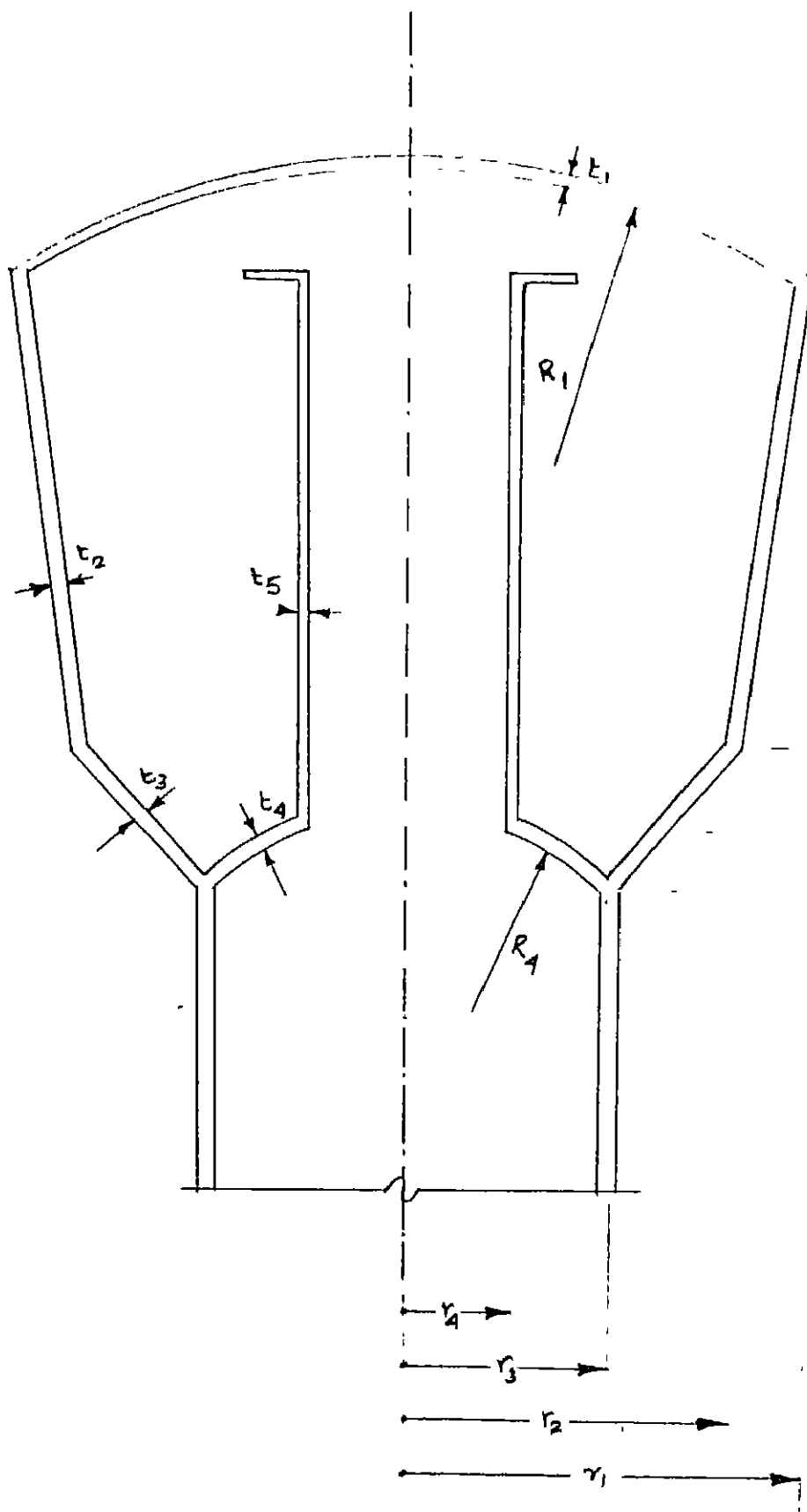
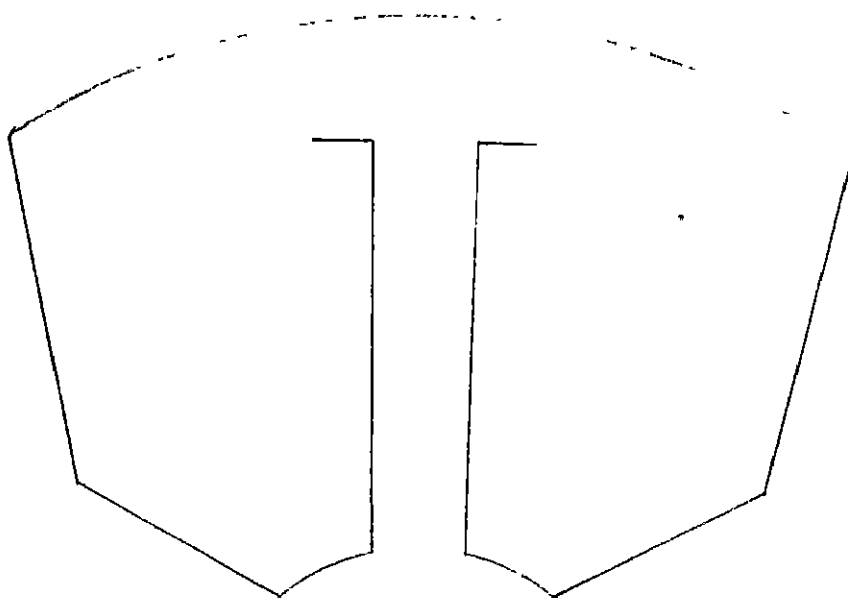
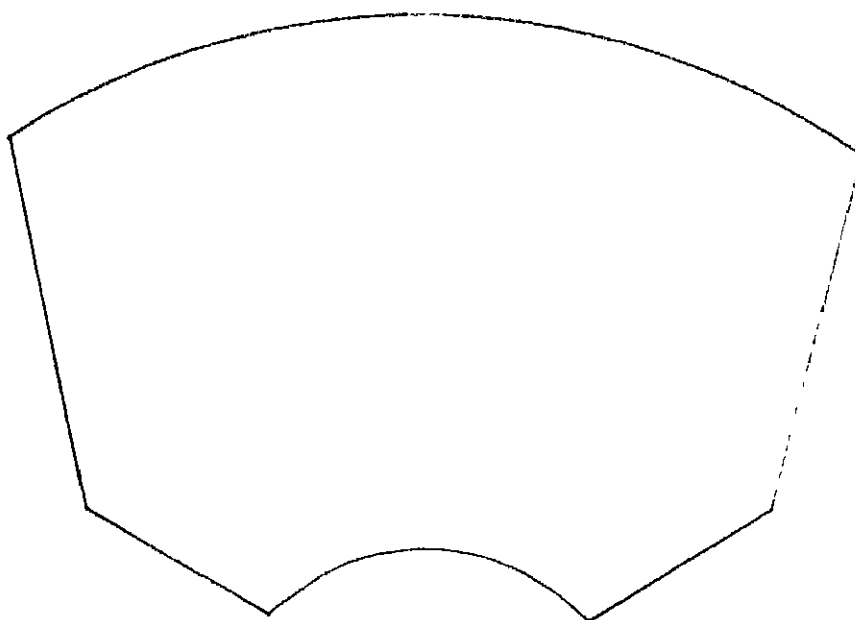


Fig.-25 SECTION OF A TYPICAL
INTZE TANK

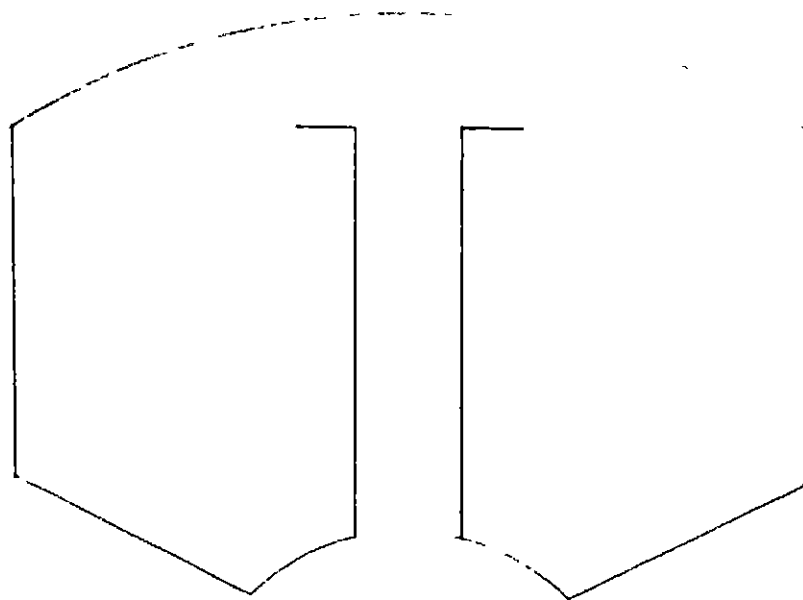


Case-1: All values are varying

Fig-26A - SECTION FOR OPTIMIZATION

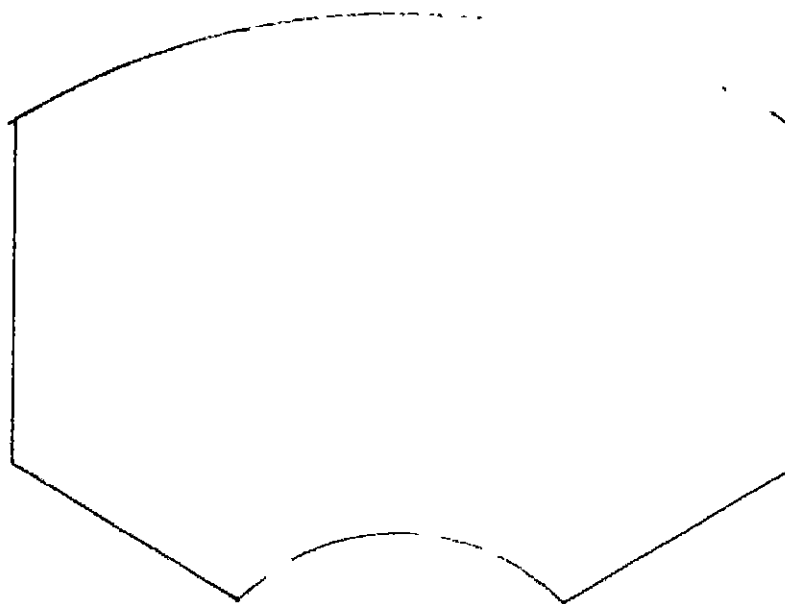


Case-4: $R4=0$, $R5=0$

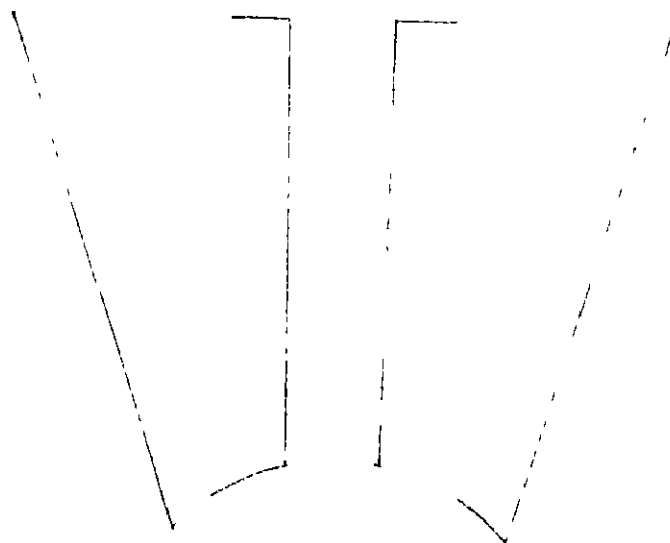


Case-2 : $R_2 = R_1$

Fig-26C - SECTION FOR OPTIMIZATION

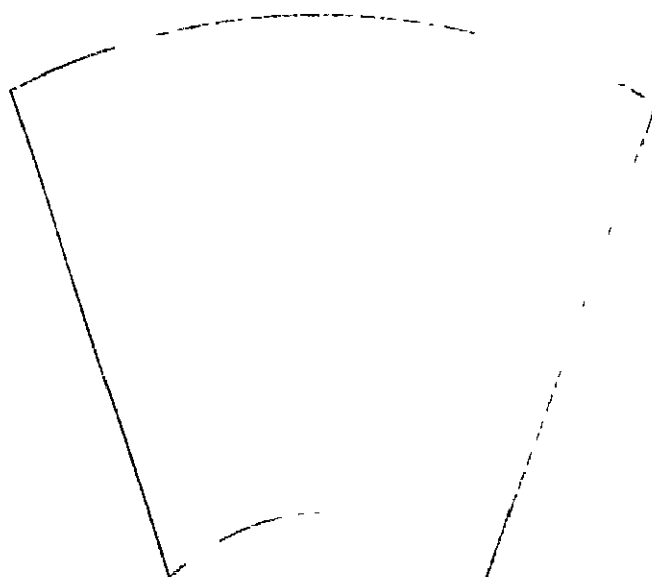


Case-5 : $R_2 = R_1, R_4 = 0, R_5 = 0$



Case - 3 : $H_3 = 0$, $R_3 = R_2$

FIG. - 26 E SECTION FOR OPTIMIZATION



Case - 6 : $R_3 = R_2$, $H_3 = 0$

FIG. - 26 F SECTION FOR OPTIMIZATION



Case-7: $R_3 = R_1$, $R_2 = R_1$
 $H_3 = 0$, $R_4 = 0$, $R_5 = 0$

Fig. 266- SECTION FOR OPTIMIZATION

5.4 DERIVATION OF ALGORITHM

The problem here has been taken as a non-linear real value programming problem. The tank has been designed sequentially from top to bottom. The weight of each part has been calculated at every stage which is added finally to have the total weight. During design procedure the values of the design parameters are regularly modified to have a realistic value in terms of construction at field.

The constraints in this optimization are mainly thicknesses and the percentages of reinforcements in the sections. The design variables for the optimization process are listed below for different cases. The table shows whether any variable have a fixed value, zero value or value equal to that of another variable.

TABLE 8 - LIST OF PARAMETERS IN SHAPE OPTIMIZATION

Variables :	R1	R2	R3	R4	R5	H2	H3
Case - 1 ::	V	V	V	C	C	V	V
Case - 2 ::	V	R1	V	C	C	V	V
Case - 3 ::	V	V	R2	C	C	V	0
Case - 4 ::	V	V	V	0	0	V	V
Case - 5 ::	V	R1	V	0	0	V	V
Case - 6 ::	V	V	R2	0	0	V	0
Case - 7 ::	V	R1	R1	0	0	V	0
V : Variable parameter, C : Fixed parameter							

Though this is a constrained optimization problem, it has been converted to unconstrained optimization problem by incorporating the thickness and reinforcement constraints during the designs of the sections. For this the following algorithm has been used.

1) Start with an intermediate value of thickness and reinforcement.

2) Call subroutine for section design.

3) If $T > T_{max}$ return to the main subroutine and changes the design variables and start the procedure again, else go to the next section.

4) If $P_{st} > P_{min}$ go to step (5) else go to step (6).

5) If $T > T_{min}$ reduce T and go to step (2) else write $T = T_{min}$, $P_{st} = P_{min}$ and go back to the main subroutine.

6) If $P_{st} > P_{max}$ increase T and go to step (2) else go to the next step.

7) If $T > T_{min}$ write T & P_{st} and go back else write $T = T_{min}$ & P_{st} and go back to the main subroutine.

For this unconstrained optimization problem "Direct Search Method" has been used. This is because, it does require derivative of the objective function. It will be a difficult job to have derivative for this complicated problem if not impossible because it deals with a number of sections. For the optimization procedure "Pattern Search Method" have been following the "Hookes & Jeeves" method / 10 /.

5.5 STUDY OF RESULTS

For an arbitrary value of m and n the programme calculates diameter and the height of of the sections. For absurd value the number of iterations will be more otherwise it calculates the minimum weight within 20 iterations.

Another problem that developed was if the circle of the second central section is not being restricted then sometimes the radius $R2$ becomes less than radius $R3$.

The results of the programme suggests that 1) upto a diameter 400 mm tanks of case 7 may be used and above this value the section becomes uneconomical 2) above this and upto a diameter of 200 mm tanks of case 6 and case 3 may be used 3) above this other cases should be used.

In case of a different $R1$ and $R2$ value in case of case 1 & case 4 the $R1$ and $R2$ value becomes nearly equal after a seven or eight iterations.

The weight of concrete to weight of water ratio varies between 0.86 and 1.03.

5.6 CONCLUSION

In engineering design two basic aspects are optimization and aesthetics, where "optimization" will consider the problem of safety serviceability etc. and "aesthetics" takes care of the artistic value of the structure.

The aesthetic rationale of design is to have a well balanced & harmonious section of a structure with a sense of beauty / 17 /. The design should have a symbolic image also. It should represent the motif of historical background, traditional culture. It should be environmentally compatible with the natural features of the location.

But when the design comes to a designer, he or she must use the common sense for the design and not merely follow some rules.

5.7 SCOPE OF FUTURE STUDY

1) A dynamic programming may be made for the whole structure for weight minimization considering a tower supported on a central stem.

2) A Cost optimization programme may be developed for water tower design.

BIBLIOGRAPHY

- 1 Anchor, R.D. (1981) DESIGN OF LIQUID RETAINING STRUCTURES - Surrey University Press - LONDON.
- 2 Dayaratnam, P. (1983) DESIGN OF REINFORCED CONCRETE STRUCTURES - Oxford & IBH Publishing Co. - NEW DELHI.
- 3 Green, J. Keith & Perkins, Philip H. (1980) CONCRETE LIQUID RETAINING STRUCTURES: DESIGN, SPECIFICATIONS AND CONSTRUCTION - Applied Science Publishers Limited - LONDON.
- 4 Kapoor, M.P. & Hariharan, M. (1973) Optimum Design of Reinforced Chimneys - OPTIMIZATION IN STRUCTURAL ENGINEERING : Symposium Warsaw / Poland - Aug.21-23 :: Springer-Verlay , NEW YORK.
- 5 Kirsch, U. (1983) Multilevel Optimal Design of Reinforced Structures -ENGINEERING OPTIMIZATION : Vol.- 6, No.- 2, pp: 207 - 213.
- 6 Mallick, S.K. & Gupta A.P.(1983) REINFORCED CONCRETE - Oxford & IBH Publishing Co. - NEW DELHI.
- 7 Mallick, S.K. & Rangasami, K.S. (1976) REINFORCED CONCRETE DESIGN - Khanna Publishers - NEW DELHI.
- 8 Marcelin, J.C. & Trompette, Ph. (1988) OPTIMAL SHAPE DESIGN OF THIN AXISYMMETRIC SHELLS - Engineering Optimization - Vol.- 13, No.- 2, pp: 109-117.
- 9 Melchers, R.E. & Rozvany, G.I.N. (1970) OPTIMUM DESIGN OF REINFORCED CONCRETE TANKS - Journal of Engg. Mech. Div.: Proc. of the ASCE : Vol.- 96, No.- EM6, pp: 1093-1106.
- 10 Rao, S.S. (1987) OPTIMIZATION THEORY AND APPLICATION - Wiley Eastern Limited - NEW DELHI.
- 11 Reynolds, C. & Steedman, J. (1980) REINFORCED CONCRETE DESIGNERS HANDBOOK - Viewpoint Publication - London.

- 12 Reiss, R. (1974) MINIMAL WEIGHT DESIGN OF CONICAL SHELLS - Journal of Applied Mechanics : Vol.-41, No.- 3, pp: 599 - 603.
- 13 Rozvany, G.I.N. (1976) OPTIMAL DESIGN OF FLEXURAL SYSTEMS : BEAMS, GRILLAGES, SLABS, PLATES & SHELLS - Pragamon Press - OXFORD.
- 14 Templeman, A.B. & Yates, D.F. (1983) A Segmental Method for Discrete Optimum design of Structures - ENGINEERING OPTIMIZATION: Vol.- 6, No.- 2, pp: 145 -155
- 15 Thambiratnam, P., Thevendran, V., Lee, S.L. & Chang, S.L.(1986) MINIMUM WEIGHT DESIGN OF CYLINDRICAL WALL TANKS - International Journal of Numerical Methods in Engineering : Vol.-23, pp:1679-1691.
- 16 Thambiratnam, P., Thevendran, V., Lee, S.L. & Chang, S.L.(1988) MAXIMIZATION OF NATURAL FREQUENCIES OF CYLINDRICAL SHELLS - Engineering Optimization - Vol.- 13, No.- 2, pp: 85-99.
- 17 Thompson, J.M.T. & Hunt, G.W.(1974) Dangers of Structural Optimization - ENGINEERING OPTIMIZATION : Vol.- 1, No.- 2, pp: 99 - 110.
- 18 Toshiaki, Ohata, Takahashi, Nobuo & Yamane, Tetsu (1981) AESTHETIC DESIGN FOR BRIDGES - Journal of the Structural Division : American Society of Civil Engineers - Vol.- 113, No.- 8, pp:1678-1691.
- 19 Watt, S.B. (1978) FERROCEMENT WATER TANKS & THEIR CONSTRUCTION - Intermediate Publishers - LONDON.
- 20 Weaver, William & Gere, M. James (1986) MATRIX ANALYSIS OF FRAMED STRUCTURES - C.B.S. Publishers and Distributors - NEW DELHI.
- 21 William, B. Kays (1986) CONSTRUCTION OF LINING FOR RESERVOIR, TANKS AND POLLUTION CONTROL FACILITIES - John Wiley & Sons - NEW YORK.
- 22 _____ INDIAN STANDARD CODE OF PRACTICE FOR PLAIN AND REINFORCED CONCRETE (Third Revision), IS : 456-1978, Bureau of Indian Standards - NEW DELHI.

- 23 _____ INDIAN STANDARD CODE OF PRACTICE
FOR STRUCTURAL SAFETY OF BUILDINGS
: LOADING STANDARDS, IS: 875-1964,
Bureau of Indian Standards - NEW
DELHI.
- 24 _____ INDIAN STANDARD CRITERIA FOR
EARTHQUAKE RESISTANT DESIGN OF
STRUCTURES (Fourth Revision), IS:
1893 - 1984, Bureau of Indian
Standards - NEW DELHI.
- 25 _____ INDIAN STANDARD CODE OF PRACTICE
FOR CONCRETE STRUCTURE FOR STORAGE
OF LIQUIDS (Part I - General
Information), IS: 3370(I) - 1965,
Bureau of Indian Standards - NEW
DELHI.
- 26 _____ INDIAN STANDARD CODE OF PRACTICE
FOR CONCRETE STRUCTURE FOR STORAGE
OF LIQUIDS (Part II - Reinforced
Concrete Structure), IS:3370(II) -
1965, Bureau of Indian Standards -
NEW DELHI.
- 27 _____ INDIAN STANDARD CODE OF PRACTICE
FOR CONCRETE STRUCTURE FOR STORAGE
OF LIQUIDS (Part IV - Design
Tables), IS: 3370(IV)-1965, Bureau
of Indian Standards - NEW DELHI.
- 28 _____ INDIAN STANDARD CODE OF PRACTICE
FOR EARTHQUAKE RESISTANT DESIGN
AND CONSTRUCTION OF BUILDINGS
(First Revision), IS:4326- 1976,
Bureau of Indian Standards - NEW
DELHI.
- 29 _____ INDIAN STANDARD CRITERIA FOR
DESIGN OF RCC STAGING OF OVERHEAD
WATER TANKS, IS:11682-1986, Bureau
of Indian Standards - NEW DELHI.
- 30 _____ DESIGN AIDS TO IS:456-1978, Bureau
of Indian Standards - NEW DELHI.

Thesis
628.13
K5270

A104195

104195

CE-1908-M-KHA-ON